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Developments in Design of Bituminous Mixtures and Pavements in Western United States

By

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I have been asked to discuss bituminous pavement practice in the far western states, and it is a little difficult to know just how to present the subject as it would seem appropriate to make comparisons and contrast western practices with those that are well established elsewhere. However, in order to do this properly one would have to be equally familiar with conditions, methods and concepts in many regions and as my own experience has been confined to one of the most western of the states I can only undertake to describe some of the methods employed and give an outline of the development and experience with examples of both success and failure. I will discuss the test methods and control procedures that have been set up in order to guard against the various defects and deficiencies that have been noted throughout the years.

The use of asphalt on highways in the western states probably began in California. The scanty record seems to indicate that some usage was contemporary with the beginning of asphaltic

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paving in the eastern states. According to Peckham, natural sand asphalts were in use in California prior to 1866 and asphaltic road oils were first used on the oiling of Ortega Hill near Summerland in the year 1894 and from that time on the use of asphaltic materials developed slowly and intermittently including paving of streets and highways with sheet asphalt, Topeka and asphaltic concrete and in the late 20's the oil mix type appeared and spread rapidly over the highways in all the western states. About seven-eighths of the paved roads in California now have asphalt surfaces of one type or another and the percentage is probably even higher for the balance of the West.

With the advent of the state highway system in 1912, the first asphaltic pavements on California highways were patterned after eastern practice, using sheet asphalt on concrete bases. This soon changed to portland cement pavements 15' wide and 4" thick. With the rapidly expanding traffic, it soon became evident that funds were not available for the construction of so-called permanent pavements and during World War I many of these pavements, including both portland cement and asphaltic types developed serious failures so that State Highway Engineer R. M. Morton concluded that it was essential to construct better bases and foundations and allow the roadbeds to become stabilized before paving. From 1915 to 1925 many miles of the highway system were surfaced with gravel in all the western states and with the rapid development of the automobile age traffic increased tremendously in the early 1920's. As a result the graveled roads were rapidly torn up and worn away to the

extent that we were losing as much as an inch of surfacing a year. Many will remember the badly wash-boarded dusty gravel roads of 1920 to 1925. And those of us who worked in highway maintenance will recall the feelings of frustration and futility in trying to keep down the dust with water sprinkling trucks operating 24 hours a day; and in attempting to maintain a smooth surface with blade graders.

The State of Oregon began in 1923 to treat these gravel surfaces with liquid asphalts of the slow curing types which were generally known as road oils (although often purchased as fuel oils to avoid higher freight rates) and following Oregon's example, California began in 1925 to try out bituminous surface applications over many miles of these older graveled roads. In the beginning, two methods were employed; one the use of relatively light road oils which was a sort of evolutionary step from the use of water. It was the philosophy that such roads would be maintained by blading while the oil would eliminate the dust nuisance. It was soon found, however, that in many cases these low viscosity liquid asphalts or road oils would cement well graded materials sufficiently to form a smooth dustless surface that did not readily form pot holes and in fact gave the appearance of a paved surface.

The second method was the so-called oil mat which was the equivalent of the present day surface treatment or "seal coat" on a well compacted or cemented gravel road.

Engineers in the southern desert regions soon realized the possibilities of surfaces that required no blading and little maintenance and in 1927, the first project was constructed in

California wherein graded aggregates and slow curing oils were mixed in place on the road to produce a substantial wearing surface 2 or 3 inches thick. The success of this pioneer section between Victorville and Barstow led to many thousands of miles of similar construction in the western states. While the grades of binder have covered the entire range of liquid asphalts including slow, medium, and rapid curing cutbacks, and emulsions the changes and developments are largely matters of detail. Following the success of the first road mix project, in 1927 the first contract was awarded requiring the use of a central mixing plant and this was followed in 1928 and 1929 by a number of similar contracts scattered throughout California and adjoining states. By the close of business in 1929, it was evident that this process could produce some excellent low cost pavements or surfaced roads, but it could also produce some outstanding and rather spectacular failures. Fig. 1.

From the first the laboratories of several states including the Materials and Research Department of the California Division of Highways began investigations which are still continuing although the scope has become expanded to embrace all types of bituminous pavements and has inevitably led into research and study of all factors and principles influencing the structural design of pavements.

The first failures observed were in the form of raveling, Fig. 1, and it became evident that this difficulty, apparently due to a deficiency in amount of asphalt, was more frequently encountered in the plant mix jobs than on those that had been constructed by the road mix method. This apparent anomaly can

be traced to the then prevailing habit of trying to judge the proper amount of oil by the appearance of the mix. With the same percentage a road mix job will look dry and a plant mix appear dark in color. In other words, if a plant mix and a road mix using the same material are brought to the same color or shade of brown the plant mix will invariably have the lesser amount of asphalt.

There was no dependable means in 1926 for determining the optimum amount of asphalt for all of the wide variety of local materials being used. It was obvious that these mixtures would not tolerate an amount of liquid asphalt corresponding to Eastern practice with sheet asphalt or asphalt concrete. Attempts were made and reported by C. L. McKesson and W. N. Frickstad(1) in 1927 to judge the correct amount of oil by means of the pat stain test. This test was never satisfactory and McKesson then proposed a formula based upon sieve analysis which became known as the McKesson-Frickstad formula.

In 1929, while resident engineer on an oil mix project, the writer developed a formula based on the idea that the amount of asphalt was proportional to the surface area of the aggregate. This formula was subsequently improved after I transferred to the Materials and Research Department. It was early evident that it was not possible to fill the voids in the aggregate with low viscosity asphalts and still have a stable mixture and the obvious effects from increasing or decreasing the fines in the grading all pointed to the conclusion that the optimum amount of asphalt was a function of the surface area of the aggregate. The surface area method(2) has been described in detail and the pro-

cedure is available for further study if desired. It may be pertinent to point out that the surface area chart, Fig. 2, reflects the idea that the film thickness (Bitumen Index) of oil or asphalt must be reduced as the surface area value is increased. This means, in effect, that a mass of small particles will not tolerate as thick a film of asphalt as will a collection of coarser materials. It does not mean, however, that the film thickness varies on each particle according to size in a given mixture. When properly mixed, all particles in the same batch tend to have the same thickness of film or bitumen index and this fact is useful when checking upon the asphalt contents by means of extraction tests. An article on this subject by A. R. Ebberts (who first coined the term Bitumen Index) is of interest[3] and is well worth reading by anyone interested in this subject.

How to secure "stability" in asphaltic mixtures was a problem very much in the minds of most paving engineers 25 years ago and the technical literature of the times contained papers by Prevost Hubbard, Hugh Skidmore, and others each of whom proposed certain types of tests which were claimed to be "stability tests" - one the Hubbard-Field[4] extrusion test and the other the Skidmore direct shear test. Many other devices have been proposed to measure this somewhat elusive property of "stability," which is a word that means many different things and has many different connotations - in asphalt pavements it implies ability to resist plastic deformation. While the Hubbard-Field test was used for several years in California in an attempt to evaluate asphaltic concrete, it was evident from the start that it could not account for the ability of pavements constructed with low viscosity road

oils to sustain heavy traffic. Samples of the most successful of the oil mix surfaces would characteristically display not over 100 and 200 lb. "stability" as compared to the 1500 or 2000 lb. thought necessary for a conventional sheet asphalt pavement. On the other hand, one sample from a badly deformed street gave 5000 lb. H.F. stability. The shear test showed similar lack of correlation with known performance. Therefore, it seemed that some test should be devised that more nearly simulated the type or direction of movement involved in the distortion displayed by unstable bituminous surfaces under traffic. The stabilometer(5) was an attempt to measure the magnitude of distorting forces in a plastic pavement under wheel loads. I believe that it is still the only stability testing device that will accommodate the entire range of asphaltic binders or aggregate gradation and indicate why any or all of the various types of asphaltic mixtures may have the requisite properties to resist the distorting effects of traffic, Fig. 3, 4, 5. It was soon found that by means of the stabilometer it was possible to test all varieties of bituminous mixtures whether the binding agent was a relatively light road oil of low viscosity or any grade up to and including paving asphalts. It also became evident that this test disclosed the reasons why light oil mixtures could be made to sustain traffic. It was somewhat surprising to find that when measured in this manner and at road temperatures of 140°F the stabilometer value of a stable pavement is of the same order of magnitude for any type of mixture regardless of the grade of asphalt, Fig. 6. It was soon evident that the stabilometer principle (which was later designated in certain quarters as a triaxial shear or

compression test) primarily measures inter particle friction which is the principle variable that contributes to "stability." However, while it was found that this internal friction was about the same for stable mixtures of all types, it was also obvious that there were certain differences in the form of hardness or toughness between asphaltic concrete using paving asphalt and the so-called oil mix type where a slow curing liquid asphalt is used as a binder. In order to measure this difference, the cohesiometer(5) was developed and demonstrated that the tensile "strength" or rather the cohesive resistance to being pulled apart was in general much higher with the harder penetration grade asphalts than with the liquid types. It was also found that high dust contents resulting from the use of filler or any change in gradation that would increase the density would consistently show an increase in the cohesiometer or tensile strength values but not necessarily in the stabilometer results. From this study, it became clear why many agencies and asphalt technologists had placed great stress upon the "stabilizing" effects of fillers and likewise upon the benefits and merits of high density mixtures. These ideas came from the use of tests that primarily measure tensile strength. However, attempts to correlate cohesiometer results with overall pavement performance were definitely unsuccessful as we found samples of unstable asphalt pavements showing grooves or transverse ripples under traffic that developed very high cohesiometer values and correspondingly high values in the Hubbard-Field test, and their counterparts today show high values in the Marshall device. On the other hand, certain oil mix roads have carried heavy traffic

for many years without signs of grooving or distortion even though cohesiometer tests are very low and values in the Hubbard-Field or shear test are negligible. We may summarize at this point then and say that it has been definitely proven that satisfactory asphaltic pavements or wearing surfaces can be constructed with almost any gradation of aggregate and with maximum size of coarsest particles ranging from sand to coarse stone two or three inches in diameter. In other words, good asphaltic road surfaces have been built with aggregate gradations ranging from sheet asphalt to penetration macadam. The only consistent relationship that must be maintained is that both the viscosity and the amount of asphalt must be appropriate for each aggregate combination used.

Following the construction of the pioneer project in 1926, oil mix construction spread through the western states, but as stated before, all projects constructed in the ensuing years did not turn out equally well. For example, the State of Arizona was the first to recognize differences in behavior under the action of water. They observed that certain oil mix surfaces softened up and became muddy after the first rain(6). In fact, some sections virtually disintegrated. Julian Powers and Wayne O'Harra of the Arizona Highway Laboratory made an investigation and suspecting that some form of emulsification was taking place, in 1929 they came up with an adaptation of a standard test for steam cylinder oil and developed the so-called "demulsibility test" to identify fine materials that seem to cause softening and disintegration when the oiled road is subjected to the action of water. At about the same time, A. R. Ebberts, working

for Allegheny County in Pennsylvania, published an article entitled "The Emulsifying Effects of Asphalt Fillers"(7) and pointed out the differences between silica and limestone fillers in their ability to retain a film of asphalt in the presence of water. Ebberts applied the term "hydrophilic" and "hydrophobic" to distinguish the two classes of mineral particles. By 1929, California had experienced an almost complete failure of a 22 mile project, Fig. 1, and an application of the Arizona demulsibility test indicated that the fine material or "filler" dust secured from an ancient lake bed deposit was very poor and separated readily from the oil when shaken in the presence of water. After a study of the literature and an investigation of the entire phenomena I concluded that emulsification was a somewhat secondary consideration or "by-product," so California renamed the demulsibility test calling it the water-asphalt preferential test in order to convey the idea that what we really were measuring and wished to know is whether the material "preferred asphalt" (hydrophobic) or "preferred water" (hydrophilic). The Arizona laboratory also developed the swell test(8), having found that most of the adverse materials would develop measurable swell or expansion when compacted specimens were placed under water.

Following the work of Victor Nicholson(9) most of the western states also adopted some form of the stripping test, usually patterned after the model developed by the Bureau of Public Roads and the Asphalt Institute(10). However, while most stripping tests are applicable to coarse particles or screenings, they are not readily applied to dense graded mixtures

containing fine dust.

As stated before, road mix construction in the western states first used slow curing liquid asphalts but as the years went by the inevitable limitations and failures led engineers to try other varieties of asphalt, hoping for a greater percentage of success. Slow curing oils often gave trouble in the wetter climates. Therefore, there has been a tendency to shift to cutbacks of the medium curing or rapid curing types. A recent survey indicates that cutbacks and paving grades of asphalt are used almost exclusively in the northwestern states while in southern California, Nevada and Arizona the preference is for SC-3 or SC-4 grades which appear to give longer life in the arid regions. It has been noted that the use of volatile solvents has a tendency to accelerate the hardening of asphalts and thus there continues to be a considerable use of slow curing types in many of the western states.

After it had become evident that a wide variety of aggregate types and gradation could be successfully employed in bituminous construction, it seemed pertinent to inquire whether or not there was any such thing as a principle or law that governed the gradation of aggregates. In other words, if we were to conclude that gradation is not important or at least not critical so far as stability is concerned, it left unanswered the question of whether gradation was important for any other reason. Therefore, an extensive series of studies was

inaugurated comparing the gradation of aggregates from both successful and unsuccessful projects which confirmed the idea that a wide variety of aggregate gradations could be used successfully but that it was usually a source of trouble if fluctuations or non-uniformity developed on any given project.[11] In other words, while it is possible to use many different gradations, it is important that the grading be controlled as closely as possible on the job, Fig. 7.

Other questions, of course, arose such as the matter of surface texture to produce a non-skid surface as a safety consideration, and in order to achieve this condition a definite amount of coarser aggregate must be incorporated in the mix. Also, it was found that while high dust contents could be used theoretically and there were occasional examples where as much as 30% minus 200 had been successfully incorporated, nevertheless, the probability or likelihood of success was greatly diminished where excess fines were involved, and it was clearly evident that high dust contents produced critical mixtures. Also such considerations as workability, ease of handling and placing on the street and the question of whether or not the pavement would be permeable depended definitely upon the gradation of the aggregate. It was also noted in connection with studies of permeability that it was a question of the <u>size</u> of the pores rather than the volume of the voids, and this observation cast one more doubt upon the validity or importance of density measurements or void volumes.

Having accumulated evidence that gradation had no fundamental or predictable effect upon stability if the asphalt

content was proper for the mixture used, it seemed in order to study the factors which influence the amount of asphalt which should be used. It was early apparent that rough textured stone particles required a greater amount of asphalt for a given gradation as compared to smooth, glassy stone types such as quartz. Porous particles such as softer sandstones, shale and some volcanic rocks require even greater amounts of asphalt. In order to permit a definite evaluation of this variable, the Centrifuge Kerosene Equivalent (CKE) method was developed.(12) It was also obvious that the specific gravity of the aggregate or the volume-weight relationships would have to be taken into account as it was and is common practice to apportion asphalt in terms of the weight rather than volume. This study brought out the fact that one of the principal sources of error in bituminous mix design is the failure to take into account variations in specific gravity, Figure 8, 9, 10, 11. In this respect, design practices for asphaltic mixtures tend to lag behind the established practice for portland cement concrete. Also, one can hardly over-emphasize the hidden pitfalls that lie concealed in the apparently simple relationships expressed by a sieve analysis or the plotted grading curve, especially when the specific gravity varies between the fine and coarser particles, Fig. 11.

Having reached the stage of development where it is possible to specify suitable gradations of aggregate and with means at hand to determine very precisely the optimum amount of asphalt and then to be able to measure the over-all stability of the compacted pavement, we hoped that the problem of bituminous pavement design was well under control. However,

failures and signs of distress continued to appear, and it was obvious that a large proportion of these surface failures could be attributed to weaknesses in the base or foundation and it seemed futile to continue attempts to improve the quality of the paving mixture without making sure that the base was of adequate thickness and of proper quality to support the pavement, Fig. 12. This study led to many years of work involving the construction of test tracks and other devices in order to determine the factors that underlie the structural design of pavements.(13) This study led to the development of a design formula in which means are provided for assigning numerical values to the destructive effect of traffic, to the ability of soil and granular base materials to resist plastic flow or deformation and with allowance made for the slab strength of the different types of pavement, including base combinations. After this design procedure had been adopted, which involved discarding the inadequate CBR test, it again seemed that we might have conquered most of the design problems. However, cracking and evidence of distress continued to be manifest on certain projects even though there was no evidence of plasticity in the base or subbase and where the total thickness of construction met all theoretical requirements, Fig. 13. It seemed evident that all pavements must have definite limitations in their degree of flexibility or in the capacity to withstand continuous and repeated bending under traffic loads. Therefore, an intensive study was launched about 1950 involving the measurement of pavement deflections on many projects, attempting to determine if there was any consistent pattern or differences in magnitude of

deflection between the pavements in excellent condition and those showing distinct evidence of cracking. This study produced clear-cut evidence that there were distinct limits to the ability of any present day pavement to withstand such flexing and that fatigue failures may be developed in almost any type of pavement whether asphaltic or portland cement concrete.[14] Work on this phase is still under way and we are now encouraged to believe that the problem can be solved and that we have most of the tools necessary to give warning when soils of undue resilience will be encountered, and it is hoped that within the foreseeable future a formula may be developed that will provide a thickness, weight, strength, or flexibility of pavement that will enable the structure to survive where resilient soils and foundations are encountered, Fig. 14, 15.

Again it might seem that with these additional test methods, analytical and design tools that at long last pavement design would approach something in the nature of an exact science. However, we must now face the fact that asphaltic pavements have been known to give trouble and to have only a limited life even over excellent foundations where the mix design was beyond question, and I now refer to deficiencies in the asphaltic binder. This lack of quality in asphalts may be manifested in several ways. The most serious and perhaps most common is lack of durability where the road surface becomes dry and hard showing increased susceptibility to cracking, ravel and other evidences of disintegration. Less frequently and less serious, asphalts may vary somewhat in their ability to adhere to the aggregate and to resist the disintegrating effects of moisture. It is

also possible for asphalts that have adequate durability to present problems during construction, being slow to set up and harden on the road. Of all these possible deficiencies, however, those properties leading to lack of durability are undoubtedly the most serious.(15, 16, 17)

Many studies have been undertaken in an attempt to develop test methods which could predict the performance and durability of asphalts in service. After many years of laboratory work, California has developed test methods for establishing relative durability on a comparative basis and also for predicting the relative rate of curing or "setting up" during the construction period. However, both test methods are time consuming, require relatively expensive, cumbersome, special equipment, and are not considered practical or at least are troublesome to apply as a routine control operation. As an expedient or stopgap procedure, California has adopted revised specifications for paving grade asphalts,(17) seeking to use the older, often inadequate test methods but with specification limits tightened so as to eliminate the more obvious unsatisfactory asphalts. In these new specifications the principal reliance is placed upon the flash test which of all current tests appears to show the highest degree of correlation with demonstrated degrees of durability, Fig. 16, 17.

Thus far we have been discussing details of mixture composition, design principles, and the numerous sources and causes of failure. The development of asphaltic highway surfaces in the western states has also involved changes in construction methods and these changes have in turn had a

definite effect upon results and in some cases upon the design of the mixtures. For example, the first work where mixing was done on the road was accomplished by blade graders often followed by spring-tooth and spike-tooth harrows. There was also a rash of special mixing devices, usually in the form of multiple blade drags that were supposed to throw the material back and forth and thus accomplish considerable mixing with one pass. Most of these were discarded sooner or later for blade graders either towed or motor patrol types, and before long this field was invaded by mechanical mixers of the traveling type. Among the better known was the Barber-Greene which elevated the material from a windrow on the road bed, delivered the aggregate into a continuous type pug mill mixer and then discharged the finished mix on to the road bed. Other types employed one or two axle mixers parallel to the direction of travel and also rotating mixers equipped with transverse axles. Examples of both types are in use today.

As stated before, shortly after the first road mix job was considered to be a success, other projects were let to contract requiring the use of a central mixing plant using a typical asphalt plant of the batch type although for the first few years driers were not required. It was soon found, however, that regardless of the fact that it was presumably more convenient and less expensive to dispense with the drier, there was actually a good deal of failure and distress attributable to excess moisture in the aggregate. Also, it was soon demonstrated that the average plant could turn out a much greater tonnage of mixture if the aggregate were heated than was possible otherwise. Contractors

soon found that there was no real economy in dispensing with the drier and the final results were invariably much superior when the aggregate was heated and dried. Thus while the term "cold mix" has been used rather widely, actually there have been few, if any, cases in the last 25 years where aggregates have been mixed cold with asphalt in a central plant. With road mixing operations, of course, the work is done under ambient temperatures and the aggregate mixed at whatever temperature was developed on the road.

Summary

In summation then, western experience with bituminous road construction has indicated that it is possible to utilize a very wide variety of asphaltic materials ranging all the way from 25 penetration paving asphalt through the entire range of liquid asphalts, even to materials having a Saybolt Furol viscosity of not more than 75 seconds at 122°F. It has also been demonstrated that usable road surfaces may be constructed with almost any gradation of mineral aggregate or combinations of particles ranging in size from 3" in diameter down to dust.[11] This does not mean that all grades of asphalt are equally satisfactory, and it is not possible to use all grades with all aggregate combinations. Many mixtures have distinct limitations and many are suited for certain conditions and not for others. A few general comments may be made. For example, macadam type of construction has produced many examples of durable roads but this is the most difficult type of construction to place so as to provide a smooth riding surface. Present day ideas of standards for smoothness are tending to eliminate the conventional

macadam type, at least for surfacing, although it may still be used for base construction at certain times. Extremely fine grained materials such as sands and silts have also been successfully incorporated into dustless road surfaces. Such materials have the disadvantage of being quite critical as to asphalt content. Thorough mixing presents a problem. Fine grained surfaces make it possible to develop very smooth surface textures which may be very comfortable to ride over but may readily present a serious skid hazard.[18] It has also proved to be a difficult matter to control proportions in order to develop adequate stability in mixtures of this type. Therefore, experience in the West has led to the widespread adoption of the graded aggregate type in which the maximum size is generally around 3/4" or 1". In recent years, the general tendency is toward a reduction in this maximum size so that materials of 1/2", 3/8" and even 1/4" maximum are being employed with increasing frequency.

Beginning with the work reported by Professor Moyer in Iowa,[18] it has become rather well recognized that coarse aggregate and coarse rough surface textures are not essential for safe non-skid road surfaces. The use of macadam types and coarse surface textures has probably been most prevalent in the State of Oregon. Throughout eastern Oregon, Washington and northeastern California, basaltic dikes and lava flows are of frequent occurrence and Oregon has employed crushed basalt on many miles of highways. This crushed material develops high stability due to rough surface texture and tends to be somewhat absorbent, thus requiring a relatively high asphalt content.

There are many areas in California and other western states where disintegrated granite is available and many deposits have proved to be excellent sources of road surfacing and base material even though it can vary considerably in quality. Experienced engineers have learned that disintegrated granite or d.g. as it is called is usually most satisfactory when obtained from shallow pits or from near the surface of the ground. As the disintegration of granite is due to the breakdown and kaolinization of the feldspar, the micaceous and clay-like products of decomposition are often detrimental. These finer fractions generally have a high affinity for water and tend to be leached down to lower levels or washed down to the toe of the alluvial fans. Many Western Highway Engineers have found that the quality of D.G. deteriorates as they work deeper or farther down the slopes of deposits.

In addition to the more common types of the run-of-the-mill aggregates, such as granites, basalts, etc., there are, of course, many special or peculiar materials in the western states. In Montana and certain parts of Wyoming, there has been some success in using a peculiar local material called scoria which is extremely porous and capable of absorbing up to 25% by weight of any liquid asphalt. In California and Oregon, deposits of volcanic cinders or pumice have been used successfully although it has been necessary to use from 10% to 15% of asphalt by weight. At times, pumice deposits contain a considerable amount of obsidian or volcanic glass. Some of the roads surfaced in Yellowstone Park contained high percentages of obsidian. Some rather strange and disturbing results may develop in trying to

make an asphaltic surface when the aggregate is a mixture of obsidian and pumice, the pumice being extremely light in weight and very porous will take up tremendous quantities of asphalt, but is very stable; the obsidian, on the other hand, being of normal gravity, extremely smooth and non-porous, will take very little asphalt and naturally has an inherently very low stability. Where the proportions of these two types of aggregate vary considerably through a deposit, the problem of fixing and maintaining the correct asphalt content is almot insurmountable. Stream deposits containing mixtures of shale and quartz particles can be almost as troublesome.

Quartz gravel or even beach gravel has been used successfully and in the Imperial Valley of California, one of the only available sources for aggregates are the old beach lines around the shores of ancient inland seas that have long since vanished. These gravels are rounded, smooth and polished, and it is very difficult to provide enough asphalt to properly cement them together without, at the same time, rendering the mixture unstable. (See stability curve on left of Figure 6.)

With the passing years and heavier traffic, the use of low viscosity slow curing oils has practically ceased in road mix or plant mix application. Road mixing operations in the northern areas usually involve cutback asphalts of the medium curing type. In California and the southwest, SC-3 or SC-4 is still used. The majority of the plant mix work is now accomplished with paving grade asphalts ranging from 85 to 300 penetration.

In conclusion, it may be stated that while a wide variety of bituminous mixtures have been used in the western states the real differences are largely differences of degree rather than in principle. It is virtually impossible to distinguish a modern road surfacing constructed under "plant mix" specifications from the so-called asphaltic concrete, and it appears that materials and combinations of materials will perform according to their inherent or significant properties regardless of names, classifications or type designations.

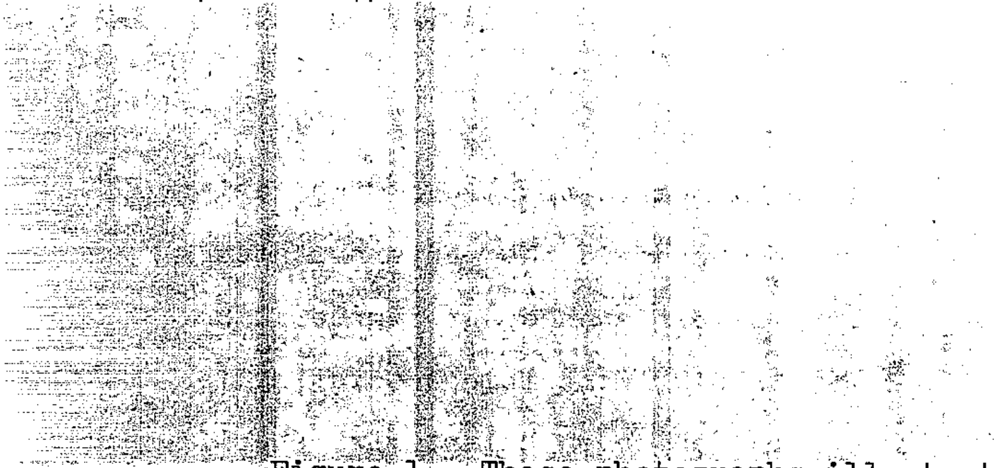


Figure 1. These photographs illustrate the disturbing and definitely discouraging failures that developed upon the third plant mix project constructed in California. This project 22 miles in length constructed in 1929 across the Mojave desert, developed failures of the raveling type throughout most of the entire length. Subsequent study and laboratory analysis indicated that a number of factors contributed. First, the pit was in a prehistoric lake bed region and each grain of sand and coarser particle was coated with a film of fine montmorillonite clay. Even tho taken from a pit in the desert where prevailing temperatures were between 100° and 120° the mix contained 3% to 4% of moisture, most of which was held in the clay films. When first placed on the street the mixture "appeared" to have plenty of oil, but as the moisture was evaporated under the hot sun the oil was absorbed leaving an insufficient amount for a mixture containing such a high dust content. The amount of dust was indicated to be 3% to 4% by dry sieving but was actually 15% to 18% by wash analysis. Primarily, these failures were caused by "too much dust and not enough oil." Secondarily, the dust and aggregate were definitely hydrophilic and failures were further accentuated by the infrequent rains.

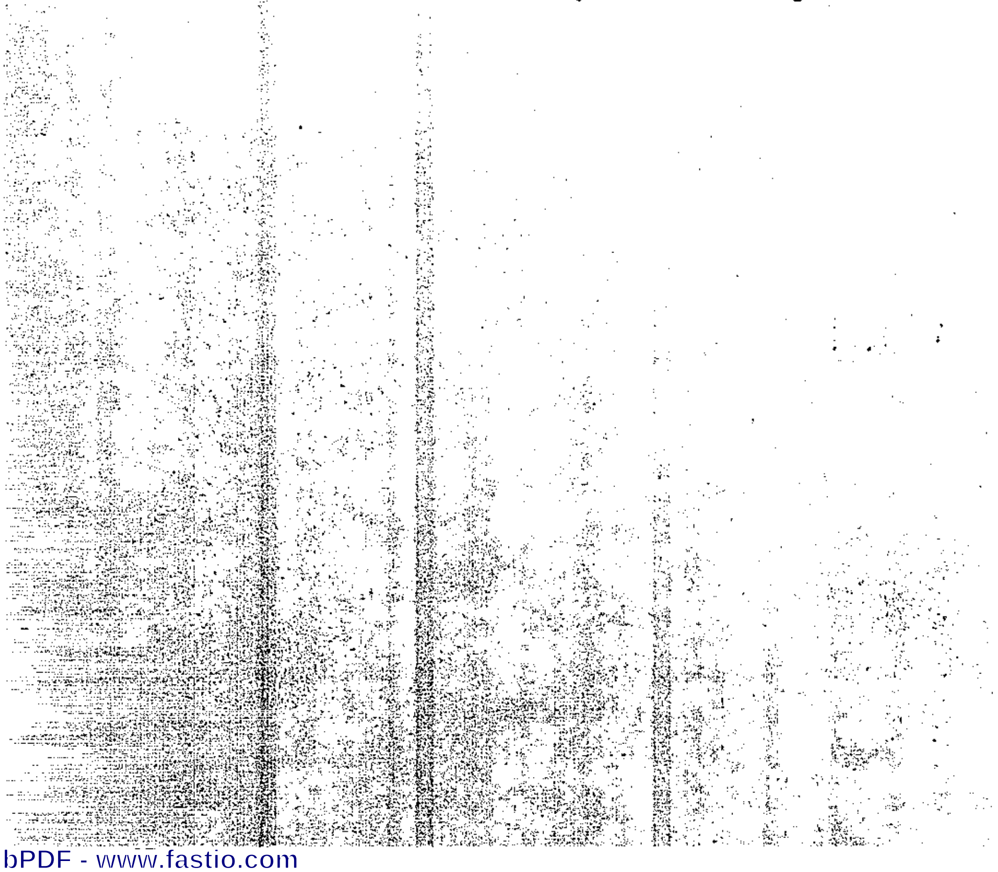


FIGURE 1

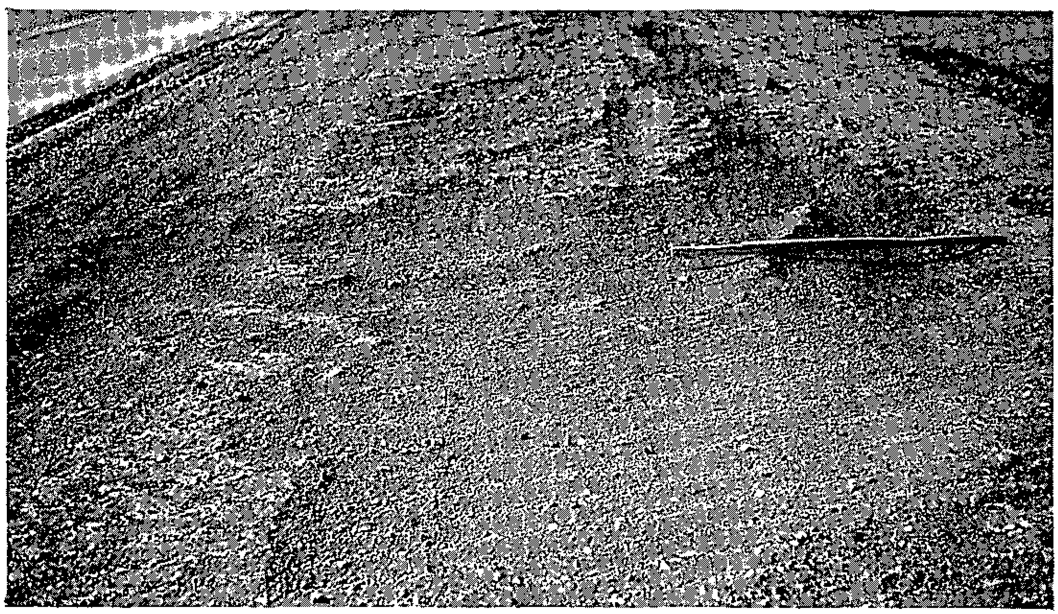


Figure 2. This chart expresses a relationship between the optimum quantity of liquid asphalt such as SC-3 or MC-3 that will be required for aggregate combinations varying in the amount of fines and varying in the surface characteristics of the particles. The abscissa scale represents the calculated surface area based on sieve analysis and particles of assumed simple geometrical shape. The vertical scale represents the amount of liquid asphalt in terms of pounds per square foot of surface area. The family of curves represents the range in particle surface roughness from very smooth, hard particles up to very rough and porous materials. A "normal aggregate" such as dense basalt or granite would ordinarily be satisfactorily represented by curve number 4 or 5.

Quartz gravel would range as low as curve No. 1 and rough vesicular lava might require curve 8 to 10.

(See Reference #2 for a more detailed discussion.)

STATE OF CALIFORNIA
DEPT. OF PUBLIC WORKS
DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

CHART FOR DETERMINING OIL CONTENT FROM SURFACE AREA OF COMBINED AGGREGATE

PROCEDURE:

1. FIND SURFACE AREA OF SAMPLE ON LOWER MARGIN OF CHART.
2. FOLLOW THE LINE UPWARD TO ONE OF CURVES.
3. THEN TO RIGHT MARGIN INDICATING BITUMEN INDEX, I.E. LBS. OF OIL PER SQ. FT. OF SURFACE AREA.
4. MULTIPLY SURFACE AREA OF SAMPLE BY THE INDICATED BITUMEN INDEX.
5. RESULT WILL GIVE LBS. OF OIL PER LB. OF AGGREGATE OR OIL RATIO.

NOTE:- NUMBERS 0-10 ON CURVES RELATE TO SURFACE FACTORS. LOWER NUMBERS APPLY TO SMOOTH HARD PARTICLES. HIGHER NUMBERS INDICATE INCREASING ROUGHNESS.

VALUES ARE FOR AGGREGATES WITH A SP. GR. OF 2.65. FOR AGGREGATES OF ANY SP. GR. CALCULATE AS SHOWN BELOW.

$$\text{OIL RATIO} = \frac{2.65}{\text{ACTUAL SP. GR.}} \times \text{SURFACE AREA} \times \text{BIT. INDEX.}$$

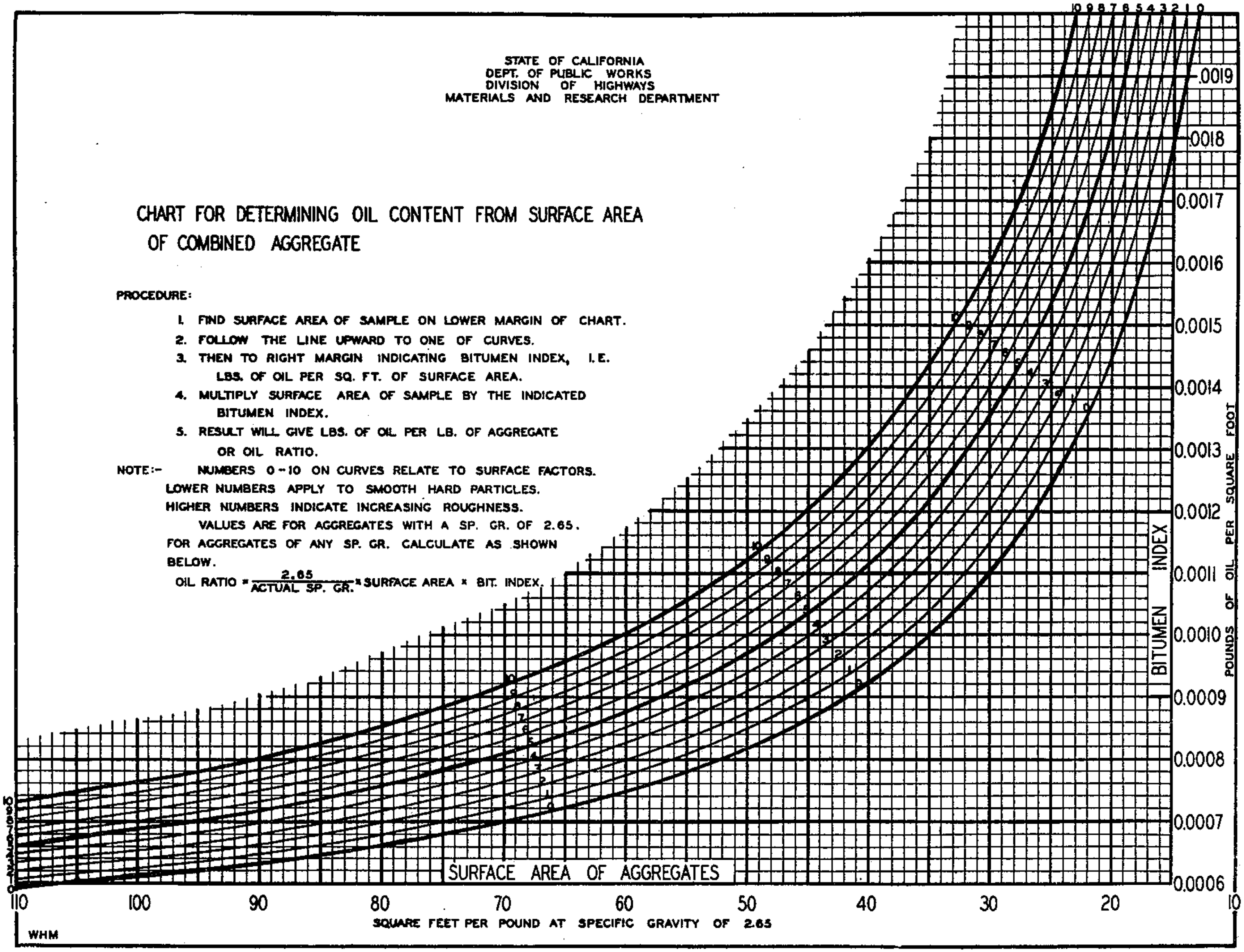


FIGURE 2

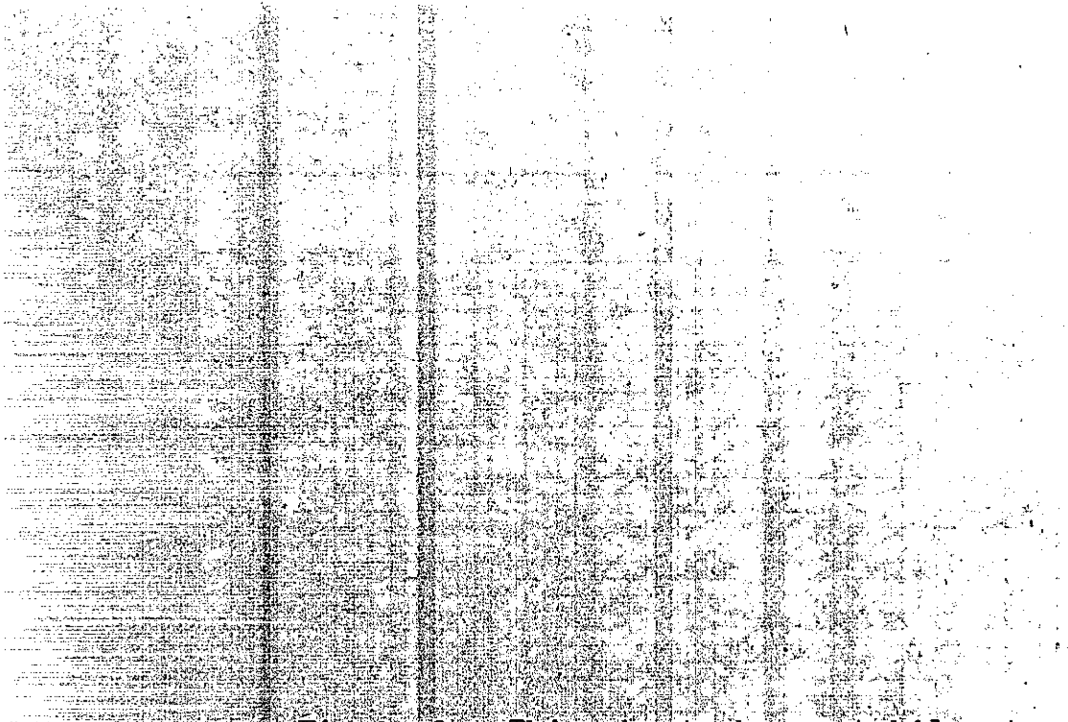
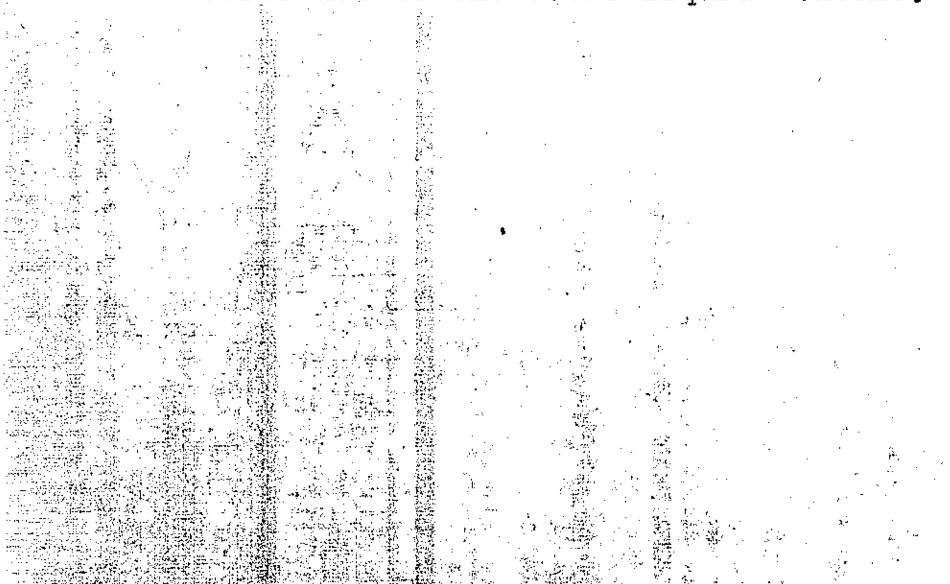


Figure 3. This chart shows stabilometer results on four samples of asphalt pavement submitted from a going project. Samples A and B represent stable mixtures that were showing no distortion under traffic. Samples C and D were showing considerable distortion. "Stability" of bituminous mixtures by the Stabilometer is ordinarily compared under a vertical pressure of 400 psi; shown on the abscissa scale. See Figure 4 for conversion of these values to a comparative stability scale plotted with reference to the total liquid content.



(See Reference #19 for a more detailed discussion.)

FIGURE 3

CHART SHOWING STABILOMETER READING (P_h)
UNDER DIFFERENT VERTICAL LOADS (P_v)

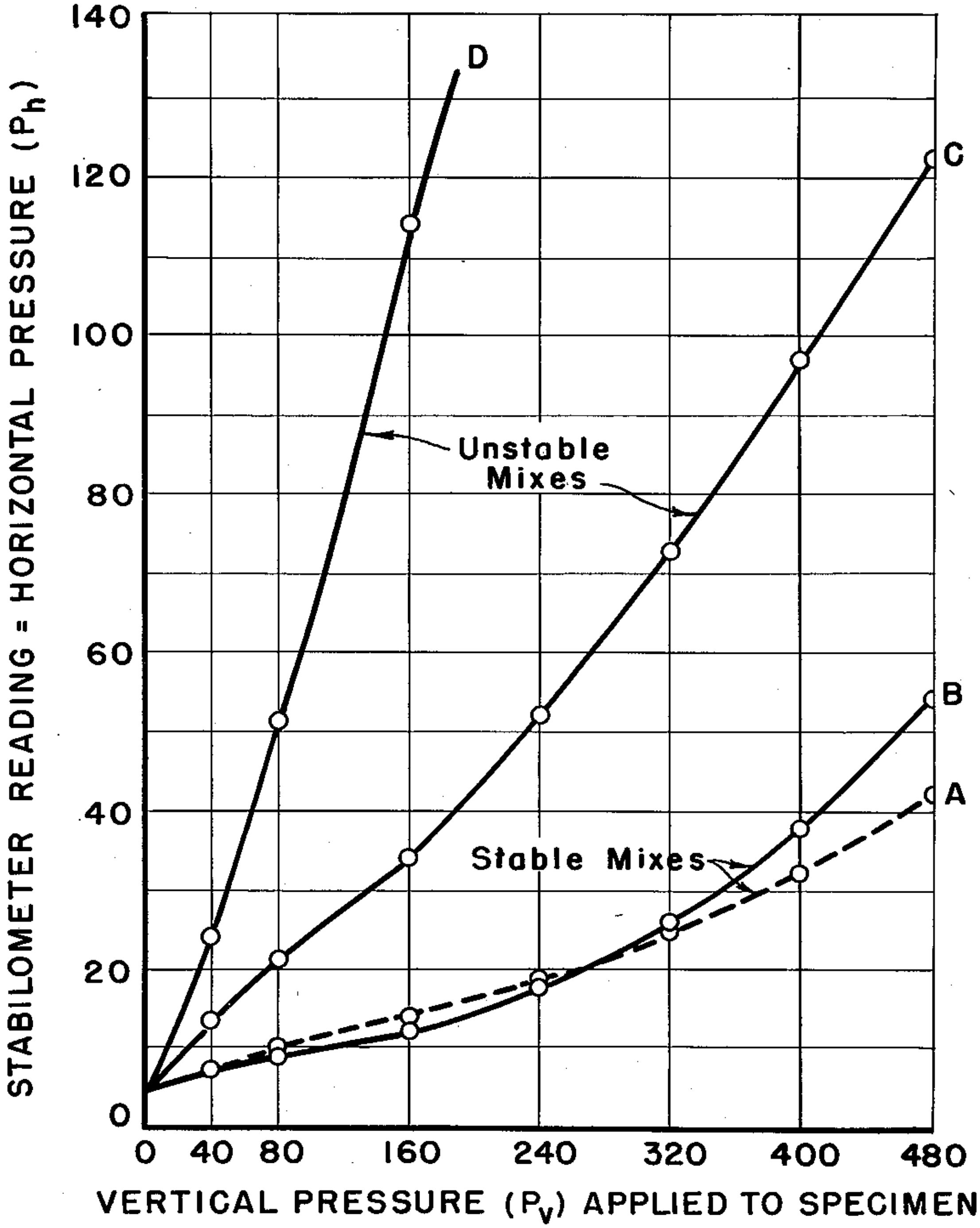
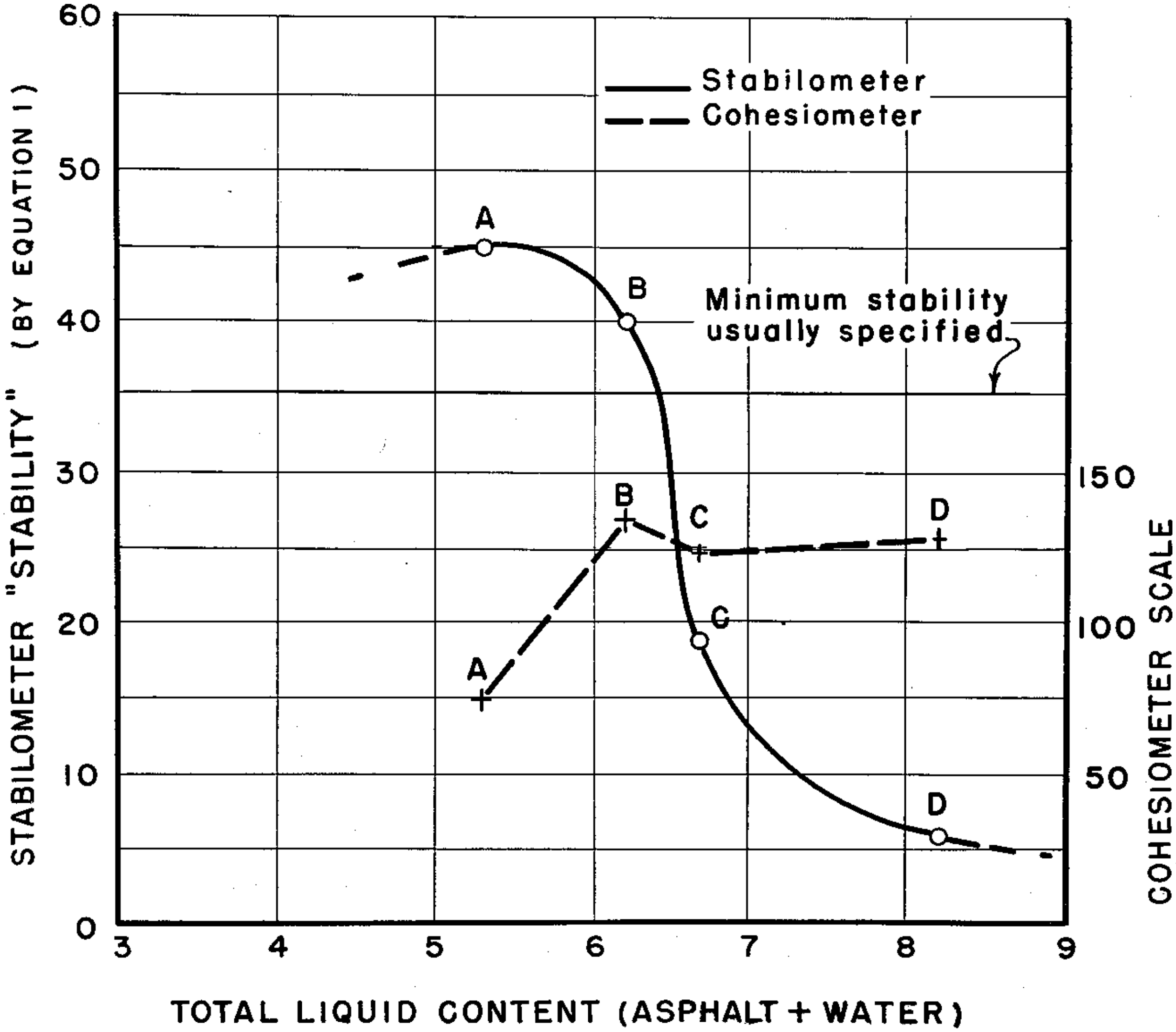


Figure 4. Plot showing same data as Figure 4 but with the stabilometer value at 400 psi plotted against the total amount of liquid, that is, asphalt plus water that was found in the specimens. It will be noted that while Sample A gave the highest value at about 5.2% of asphalt, Sample B, with more asphalt and somewhat lower ordinate, still shows adequate stability. Sample C contained about 6-1/2% of asphalt plus a little water while Sample D contained only about 6% of asphalt but had more than 2% of water present. This chart illustrates that the amounts of water and of asphalt should be considered as additive and the mixture will usually be unstable if the total amount of liquid exceeds the capacity of the aggregate. Cohesiometer values for these samples are also shown and illustrate the frequently observed fact that the best cohesion or tensile strength is often achieved at the highest liquid or asphalt content, often far beyond the amount which is optimum so far as stability is concerned. This plot illustrates the danger of replying upon test methods that primarily measure or are unduly affected by the cohesive or tensile resistance.

(See Reference #19 for a more detailed discussion.)

FIGURE 4

STABILOMETER AND COHESIOMETER VALUES ON FOUR SPECIMENS WITH DIFFERENT LIQUID CONTENT



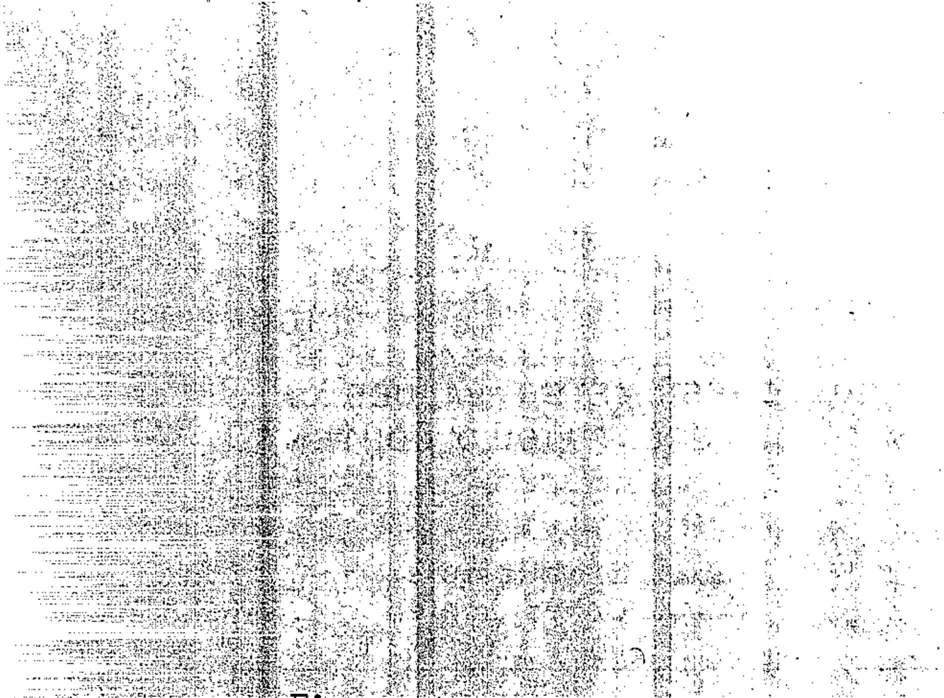


Figure 5. Photograph of an unstable asphalt surface involving both vertical and lateral deformation. Groove in wheel lane is shown by depth of depression under straightedge. Evidence of lateral movement is shown by wavy traffic stripe. Such a pavement can show fairly high values when tested by any of the methods that measure tensile strength primarily, but such a pavement invariably shows low internal friction values irrespective of the type of asphalt or gradation of aggregate.

FIGURE 5

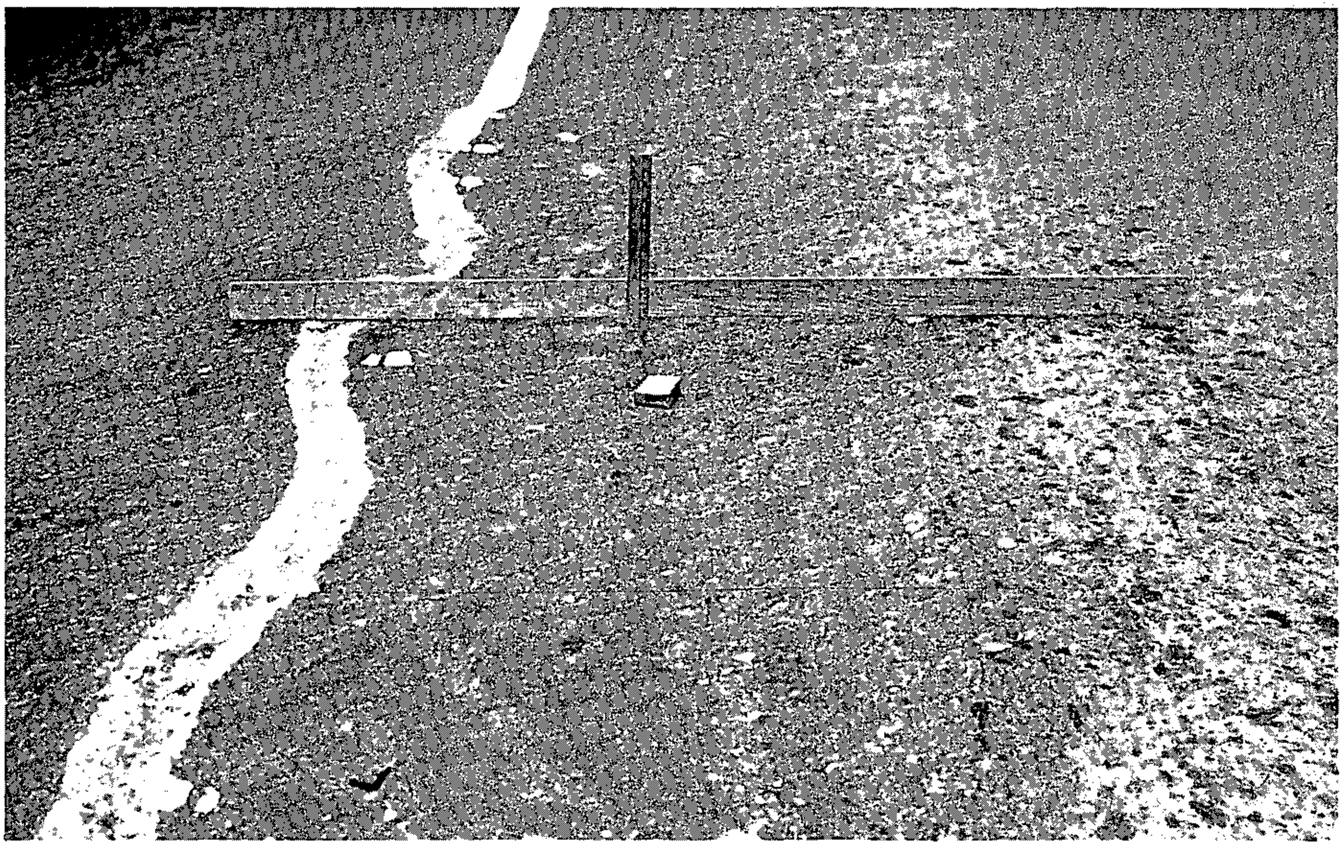


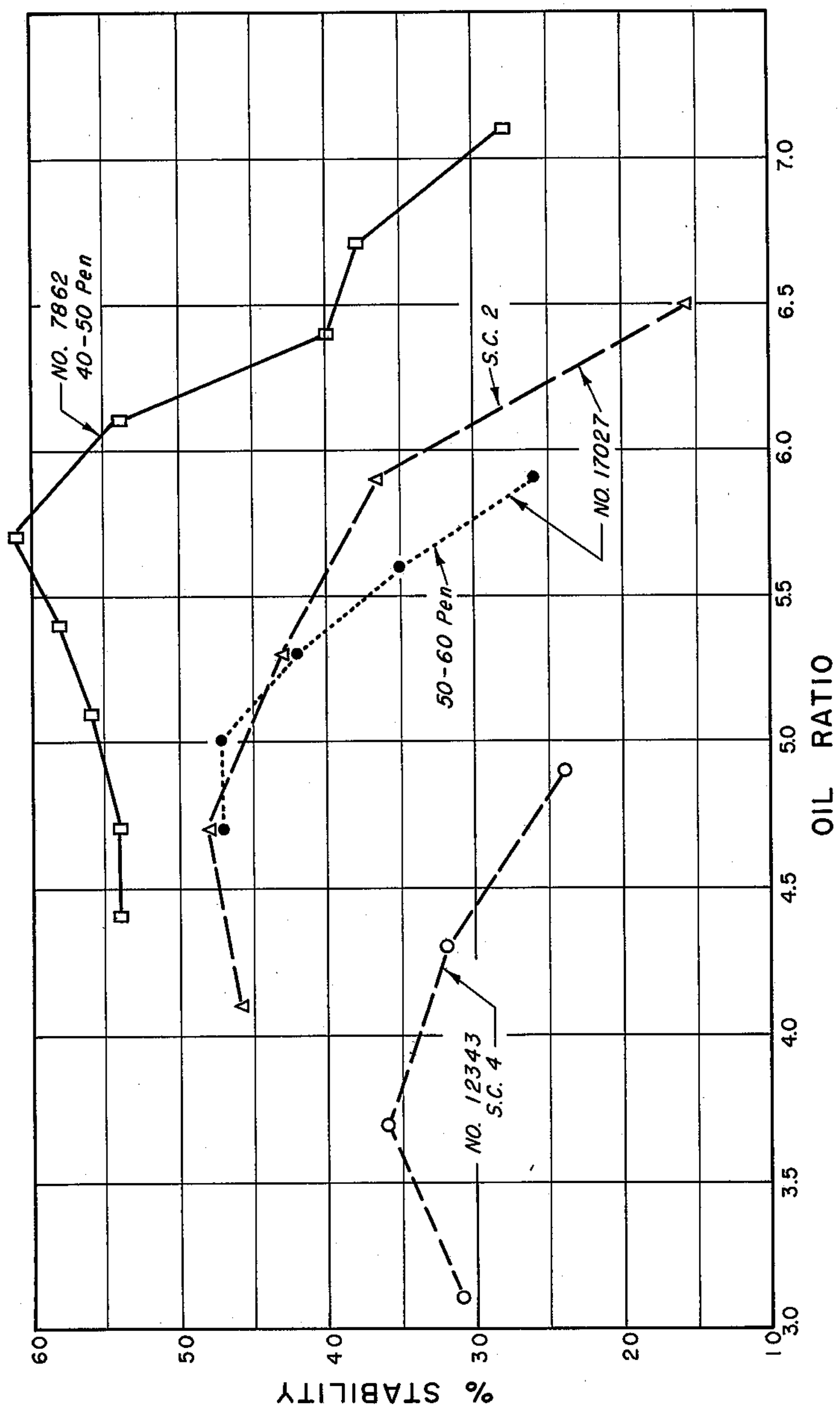
Figure 6. This is a plot showing the relationship between "stability" as measured by the stabilometer showing three different types of mineral aggregate, each tested for stability with different percentages of asphalt as shown on the abscissa scale. Sample No. 12343 is a smooth rounded gravel from an ancient beach line in the southern Imperial Valley. The curve shows that the best stability obtainable is not very high and would be barely acceptable for average traffic. This value was reduced by additional amounts of SC-4 liquid asphalt.

Sample No. 17027 is a more normal quite stable aggregate of basaltic type. Here the curve shows the close similarity in stability between SC-2 liquid asphalt and 50-60 penetration paving asphalt. Note the relatively high stability with the optimum ranging from 4.7% to 5.0%. Attention is called to the fact that stability drops faster with the addition of heavy paving asphalt than with the SC-2 road oil. This difference is not uncommon.

Sample No. 7862 is an aggregate of unusually high stability due to its very rough surface texture and moderate porosity. Note here that the optimum amount of asphalt is about 5.7%, but this amount is so close to the amount required to fill the void space that stability falls rather rapidly with higher asphalt contents although it would probably be adequate under average traffic with asphalt contents as high as 6.5%.

It should be mentioned that harder asphalts may show higher stabilometer values at times especially at lower temperatures, but in general stabilometer values do not show the differences that will be indicated by a cohesive or tensile strength measure.

FIGURE 6



NO. 7862
40-50 Pen
S.C. 2
NO. 17027
50-60 Pen
NO. 12343
S.C. 4
OIL RATIO
% STABILITY

Figure 7. This is an idealized grading chart of smooth curves drawn to indicate a satisfactory "envelope" indicating an acceptable range of tolerances. It should be borne in mind that successful pavements have been built with a wide variety of aggregate gradations in which the plotted grading curves may depart considerably both in position and shape from the one shown here. However, such gradations will often have special features or objectionable characteristics that may or may not be important on a specific project.

It is the concept that a grading curve is best viewed as simply a usable or workable "path between obstacles." If the obstacles do not exist or have been otherwise dealt with, then a correspondingly greater latitude is available for the "path of the grading curve." No "fundamental principle" or mathematical relationship has been discerned so far as the granulometric composition of mineral aggregates is concerned. Any curve can be described by a formula but there appears to be no mathematical expression that rests on sound "principles" or "fundamentals." A grading curve must be "molded" or "manipulated" to fit each set of conditions and circumstances.

Only experience will tell the engineer how best to do this.

(See Reference #11 for a more detailed discussion.)

FIGURE 7

GRADING CHART
for
BITUMINOUS MIXTURES

Maximum Size Agg. = 1"

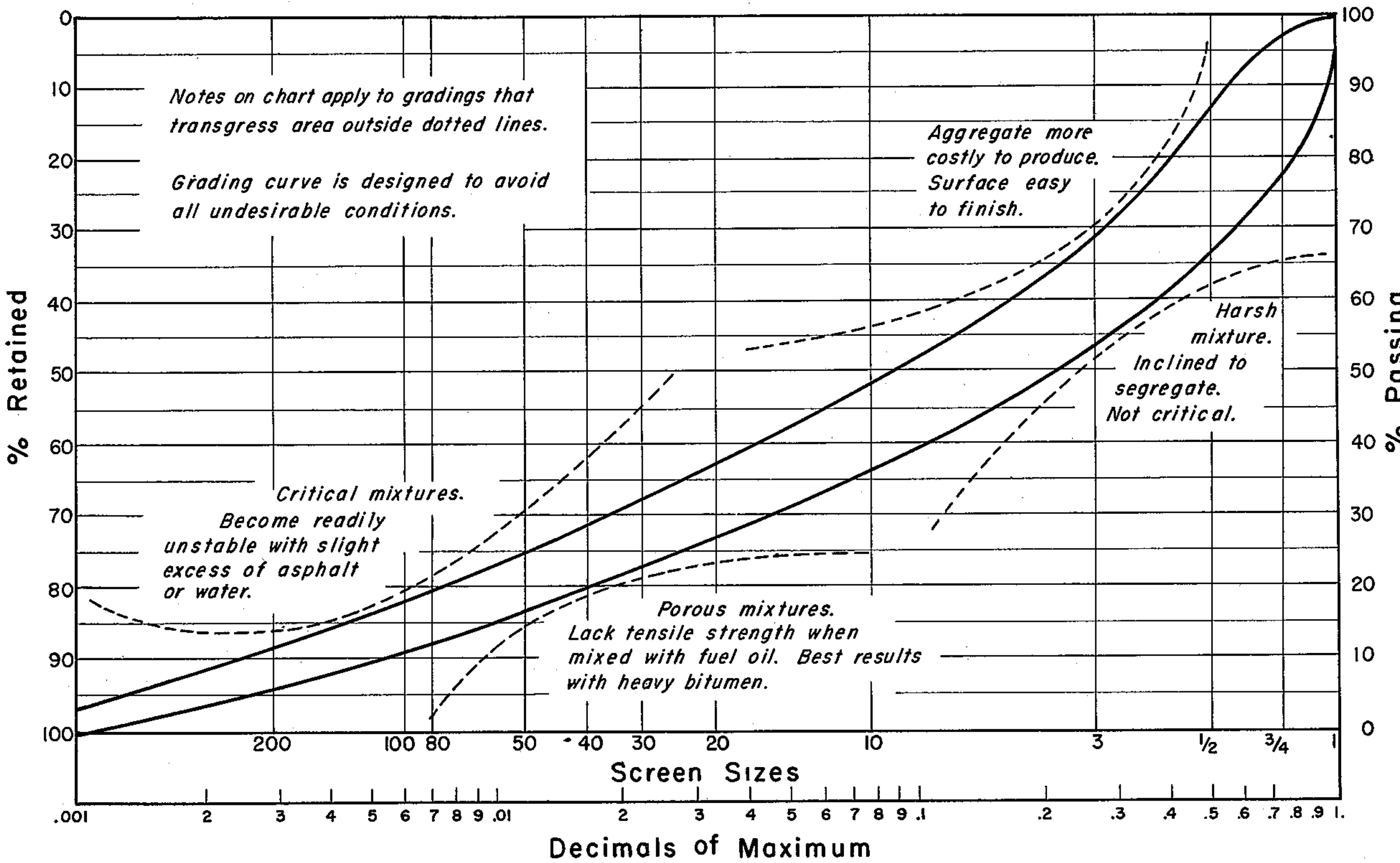


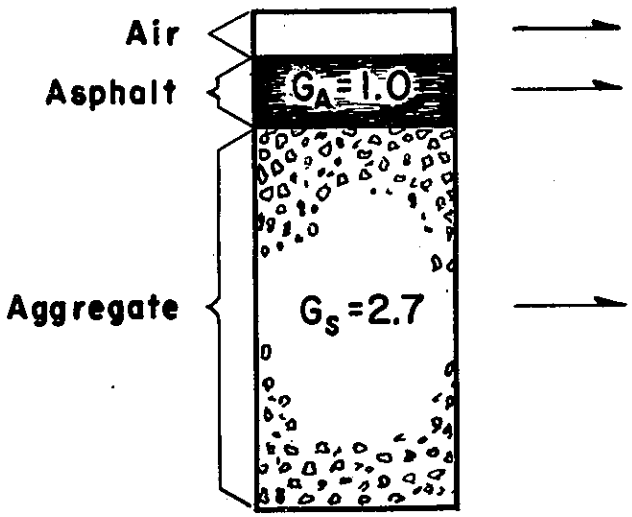
Figure 8 is self-explanatory. On the left, the diagram indicates the typical proportions by volume of the three components of an asphalt mixture; namely, the aggregate, the asphalt, and the air voids. In the tabulation under relative proportions the first column expresses the weight of asphalt and the weight of aggregate that would be used in a one ton pug mill mixer where the asphalt content was fixed at 5% by weight of the mineral aggregate (ratio of asphalt to aggregate is the basis for mix design in the western states). Column 2 shows the same figures expressed as percentage or as the asphalt ratio. Column 3 represents the same proportions where the asphalt and aggregate are expressed as percentages of the total weight of the batch. Finally, the last column indicates the relative percentages by absolute volume, assuming that the aggregate has a specific gravity of 2.7 and the asphalt a specific gravity of 1.0. It is important to remember that while weight proportioning is unquestionably the most accurate and most satisfactory method for actual field control and construction purposes, the true relationships which concern the designer must be based on the relative volumes of the three elements.

(See Reference #20 for a more detailed discussion.)

FIGURE 8

Example Illustrating Volume-Weight Relationships in a Batch of Asphaltic Concrete

Diagrammatic Composition



Compacted Weight 151 pcf

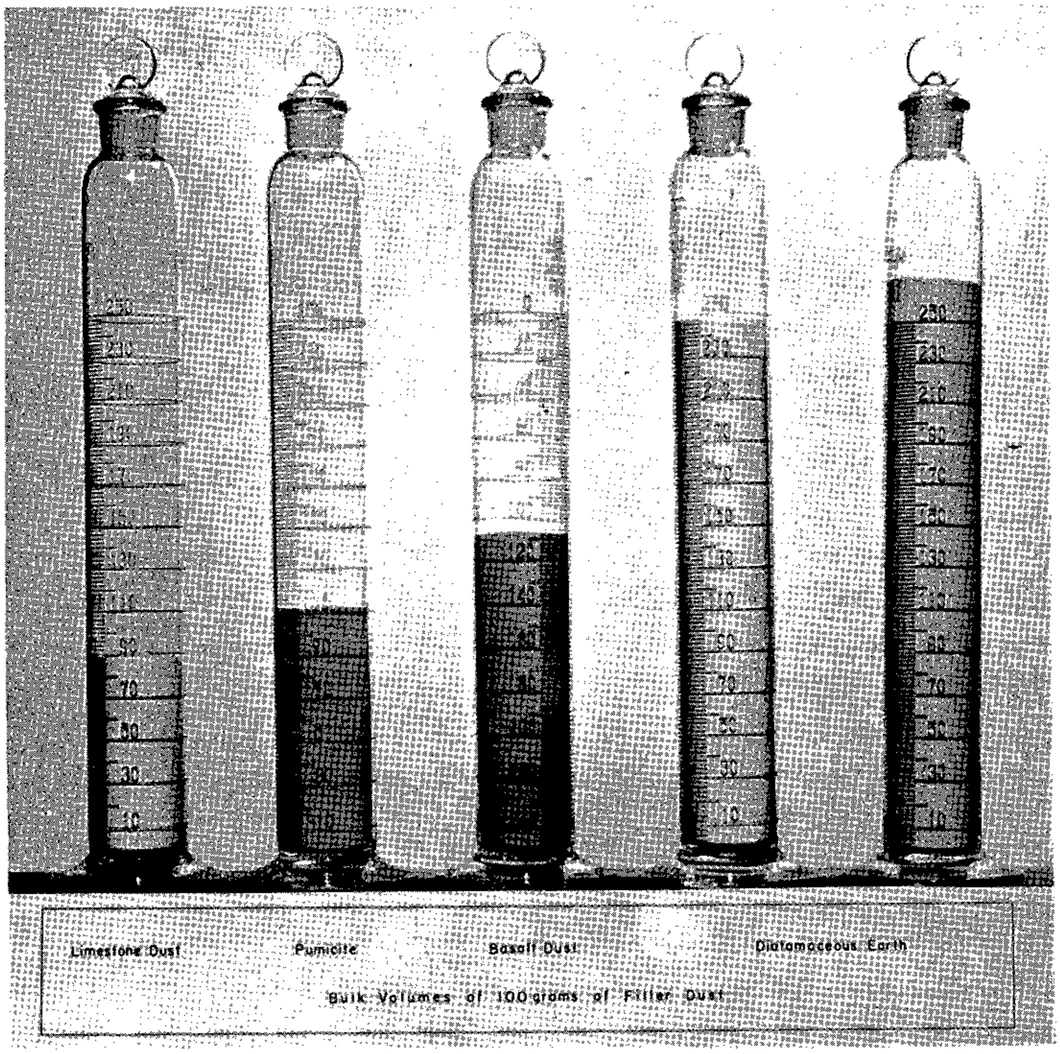
Relative Proportions			
By Weight			By Volume
Batch	% of Agg	% of Total	% of Total
0	0	0	3.1
100	5	4.76	11.5
2,000	100	95.24	85.4
2100	105	100	100

Figure 9 illustrates variations in the relationship between both volume and weight for five samples of fine rock dust or filler. On the left is a standard limestone dust. The samples on the right are diatomaceous earth. If the dust content of a dense graded mixture is found to be 6% by weight, for example, it will make a great deal of difference whether the volume of dust actually in the mixture corresponds to the graduate on the left or to one of those on the right hand side of Figure 9. Furthermore, corrections in the grading curves to compensate for abnormal volume weight relationships in the fine fractions will influence the percentages passing all other sieves in the series.

One percent of diatomaceous earth will provide as much filler as will 8% of limestone dust for example. However, the sand content passing #8 sieve will also have to be corrected downward with the result that 32% of sand including the light weight filler will give the same mix composition as 36% sand with the heavy dust. The percentages of all fractions must be corrected if any substantial differences exist in the volume-weight relationships.

(See Reference #21 for a more detailed discussion.)

FIGURE 9



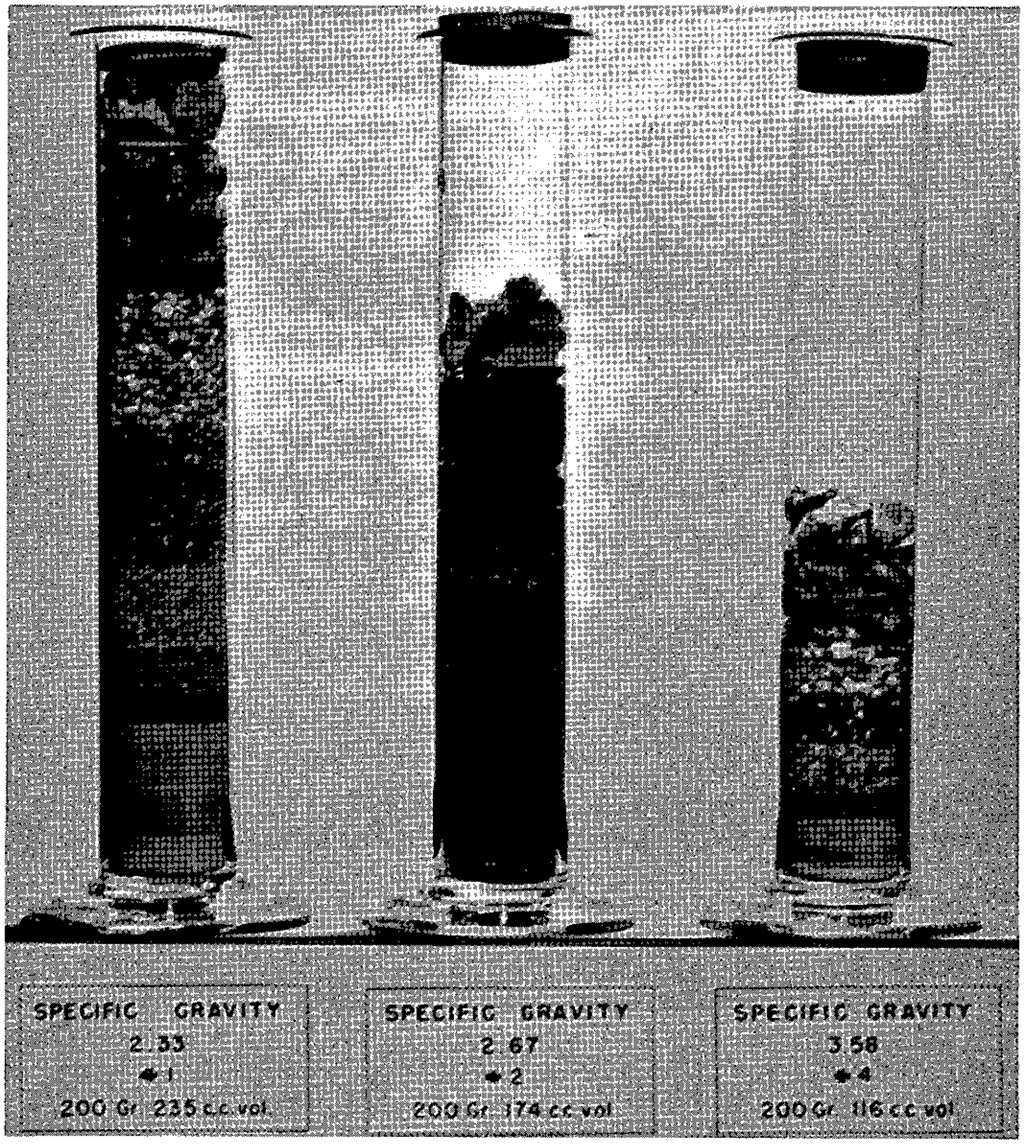
Bulk Volumes of 100 grams of Filler Dust

Figure 10 illustrates differences in volume resulting from coarse aggregates varying from specific gravity of 2.33 up to 3.58. In these graduates each fraction as separated by the standard sieves has been dyed to provide a color contrast and placed in separate layers. If the fractions were intermingled, the overall volume would, of course, be reduced in each case.

Figure 11. There are innumerable combinations which could result from mixtures of fine and coarse particles where the specific gravity varies between the separate size groups. Here are shown four examples of many possible combinations. The two graduates on the left have normal coarse aggregate and sand both having a specific gravity of 2.65. Each has 6% of filler. However, the left hand graduate contains diatomaceous earth having a specific gravity of 2.00 while the second graduate has a "normal" filler such as ground limestone rock. The third is the same as the second except that it contains lightweight coarse aggregate, and the fourth graduate contains a sand of low specific gravity. In the set of examples, the only "normal" combination is in the second graduate from the left. If we recognize that the quantity of asphalt required is a function of the particle surface area, it becomes apparent why corrections and adjustments must be made in a sieve analysis expressed as percentage of the total weight. Some marked aberrations in mix design can result from failure to adjust for differences in specific gravity.

(See Reference #21 for a more detailed discussion.)

FIGURE 10



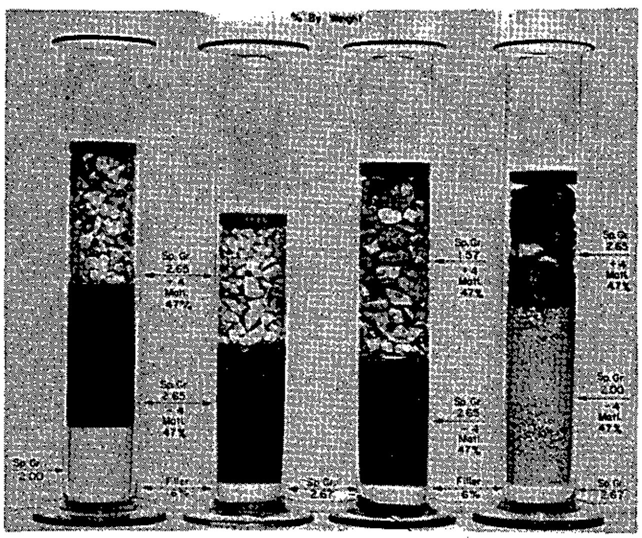


FIGURE 11

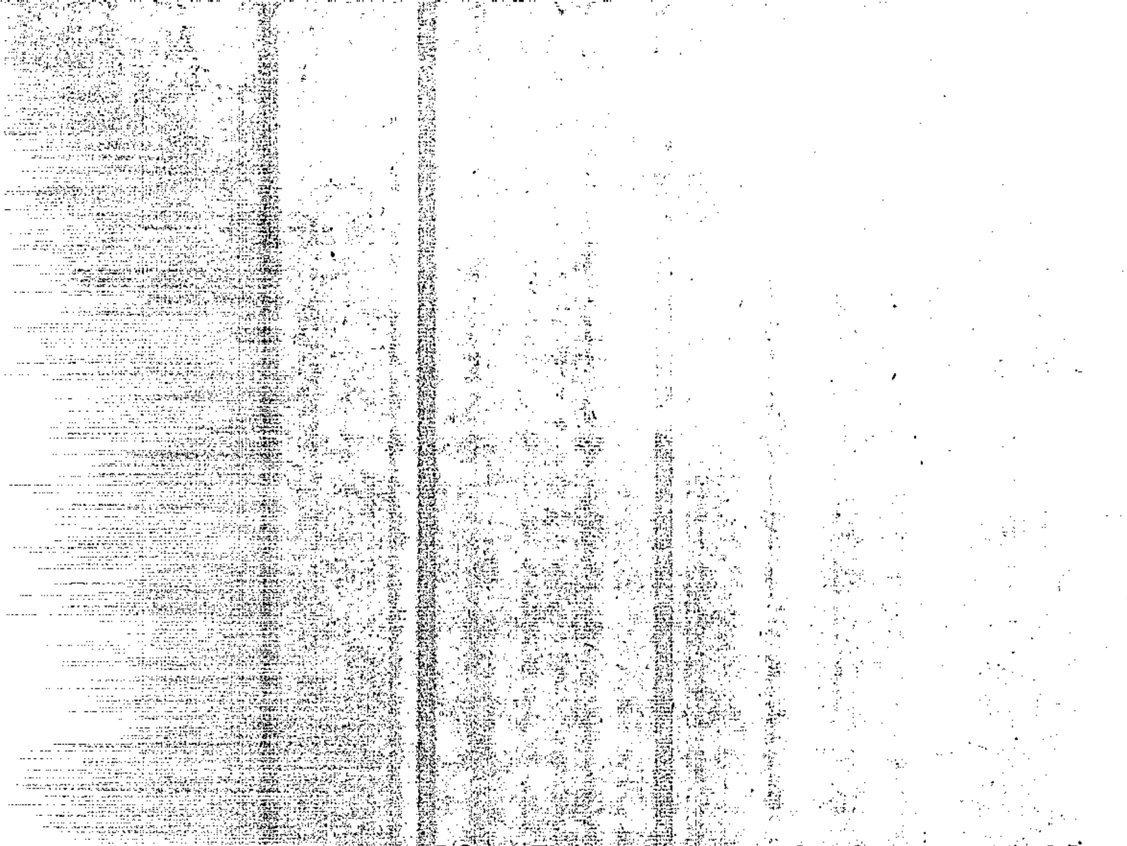


Figure 12. This photograph shows a stable asphalt surface of satisfactory composition rutting and breaking up because of unstable base or foundation.

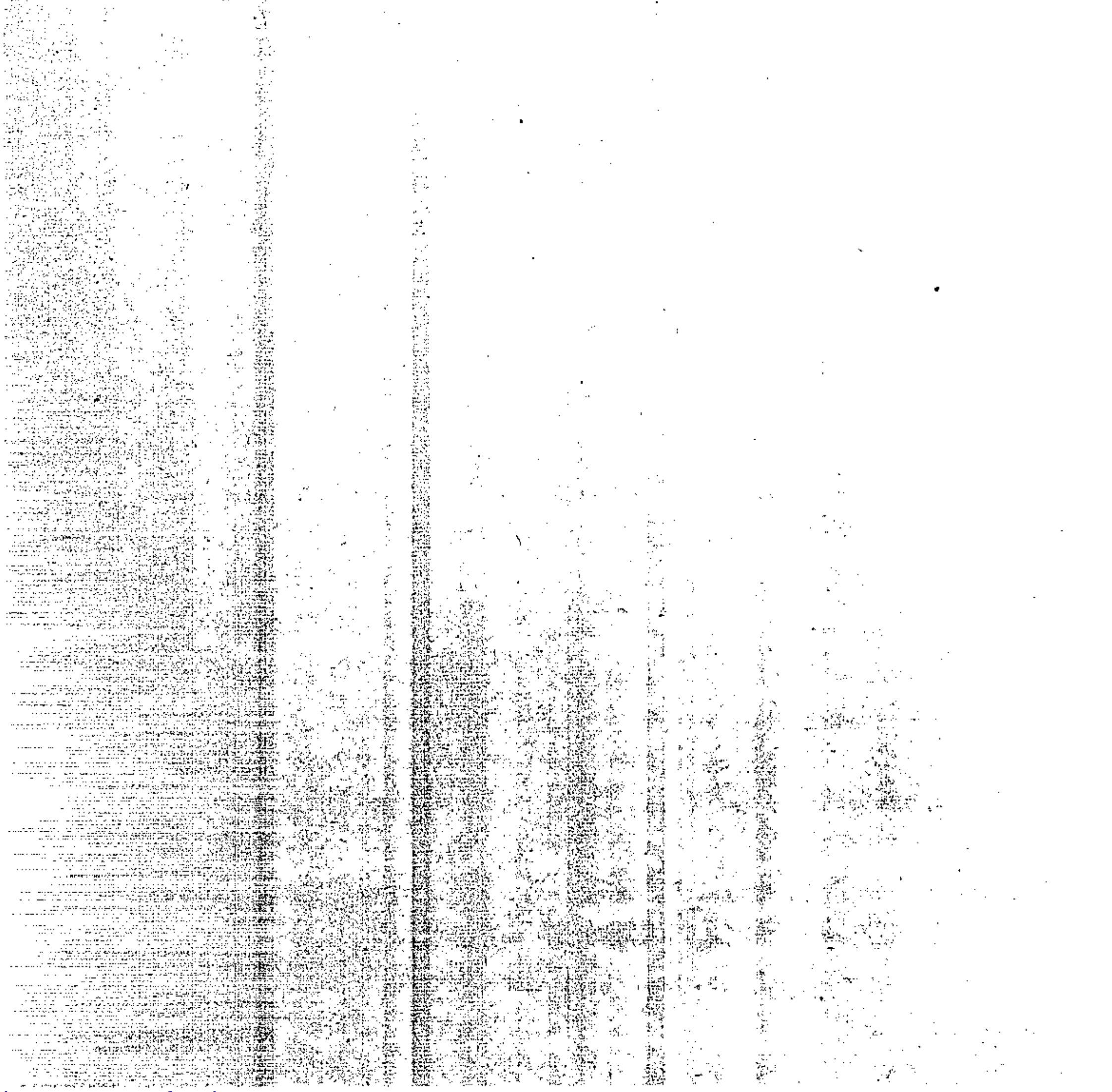
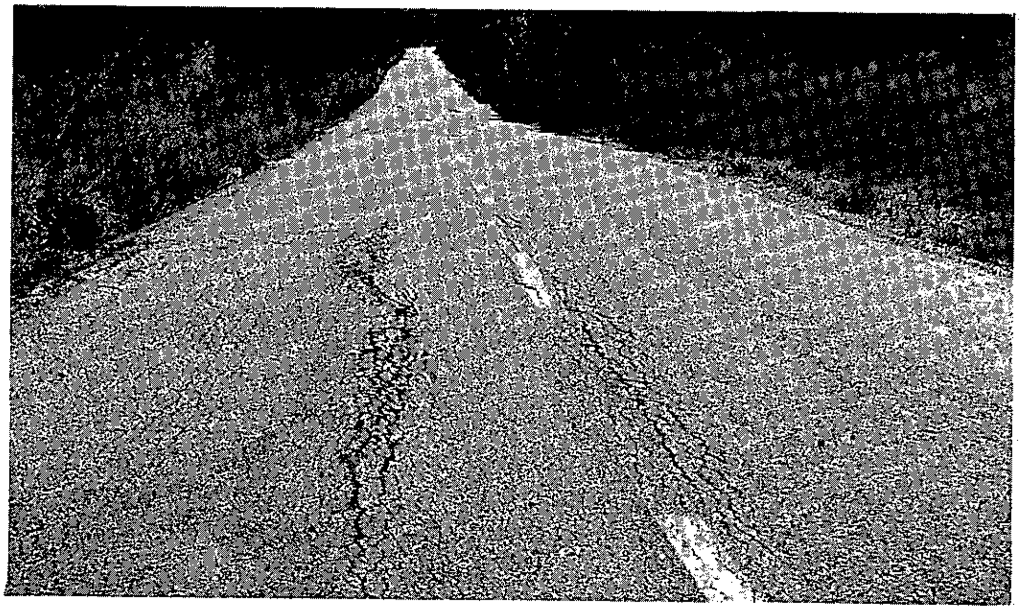


FIGURE 12



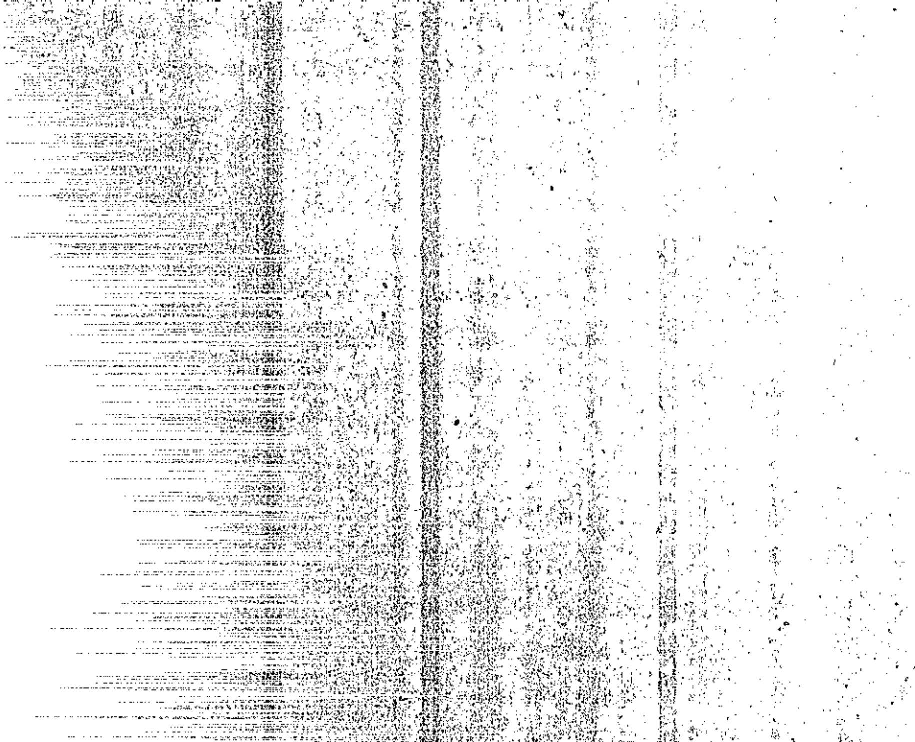
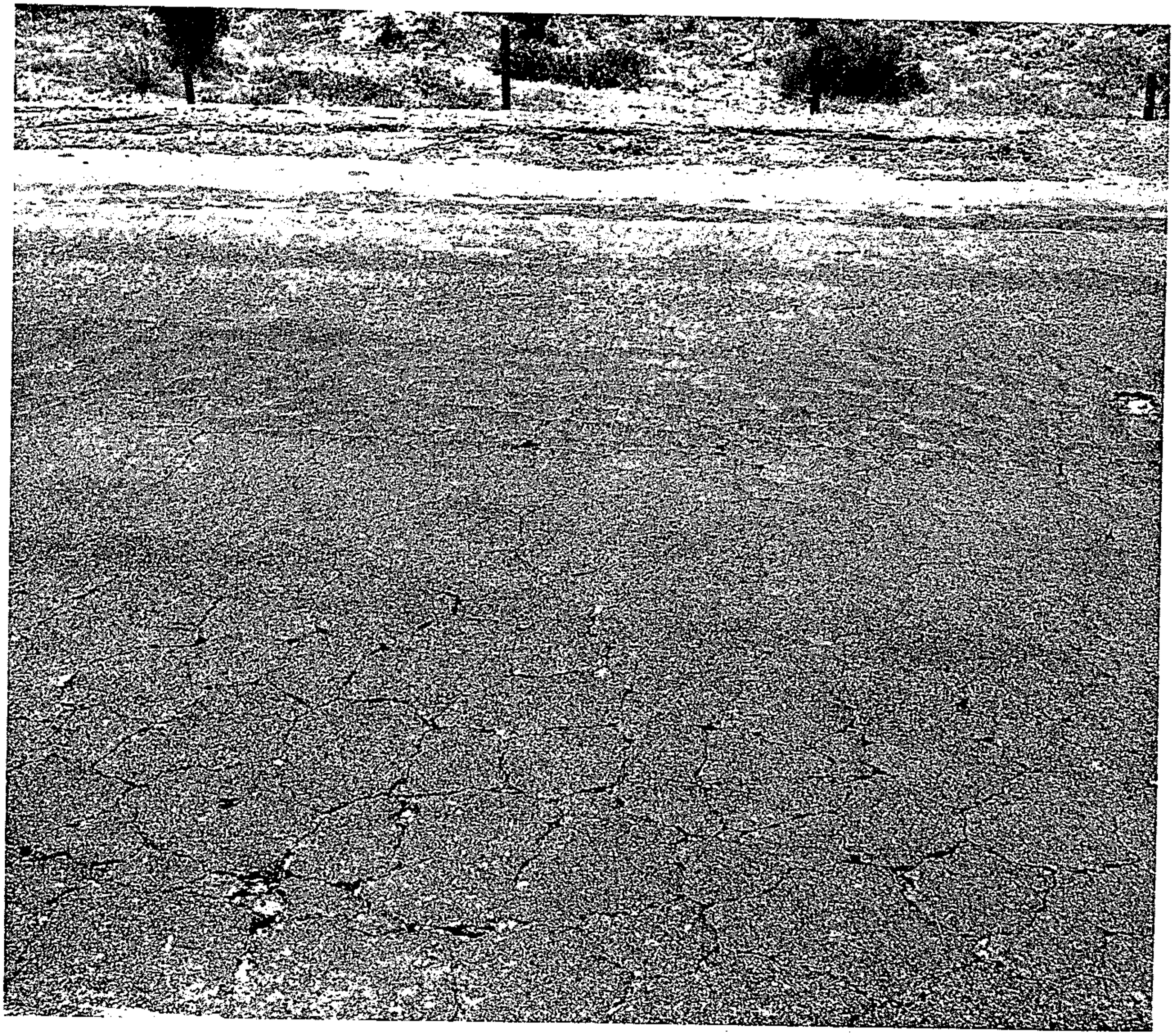


Figure 13. This is a photograph of a paved surface consisting of 4" of asphaltic concrete resting upon 8" of crushed stone under which there is a 6" layer of good granular subbase. There is no evidence of plastic deformation or lubrication and the structural design is of adequate depth so far as the ordinary concept of "bearing value" or resistance value of the soil is concerned. However, tests showed that this pavement is undergoing substantial deflection under each passing load and the undesirably hard, brittle asphalt pavement could not withstand the continued deflection developed by heavy wheel loads. The underlying basement soil was found to be unusually springy or resilient, presumably due to the presence of a large quantity of mica in the soil.

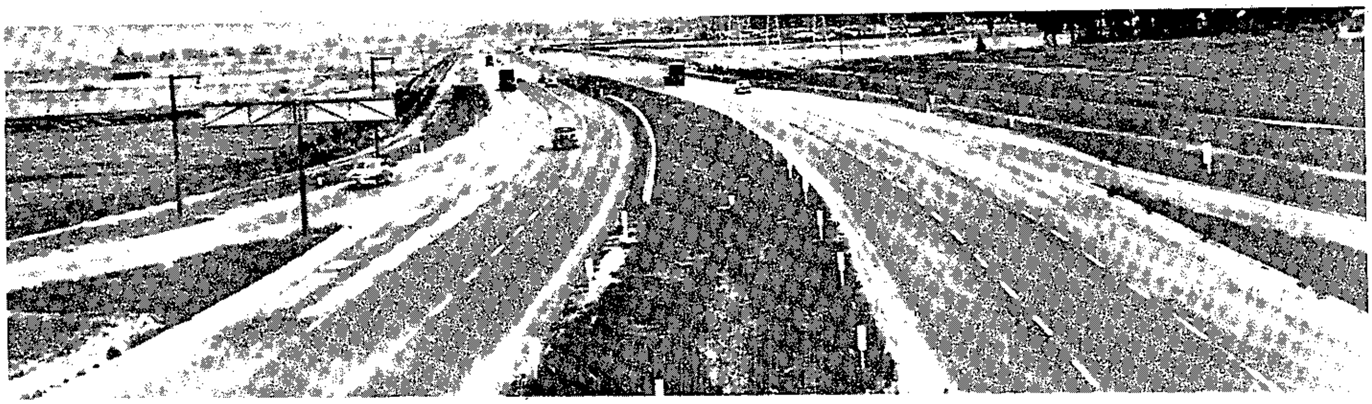
(See Reference #14 for a more detailed discussion.)



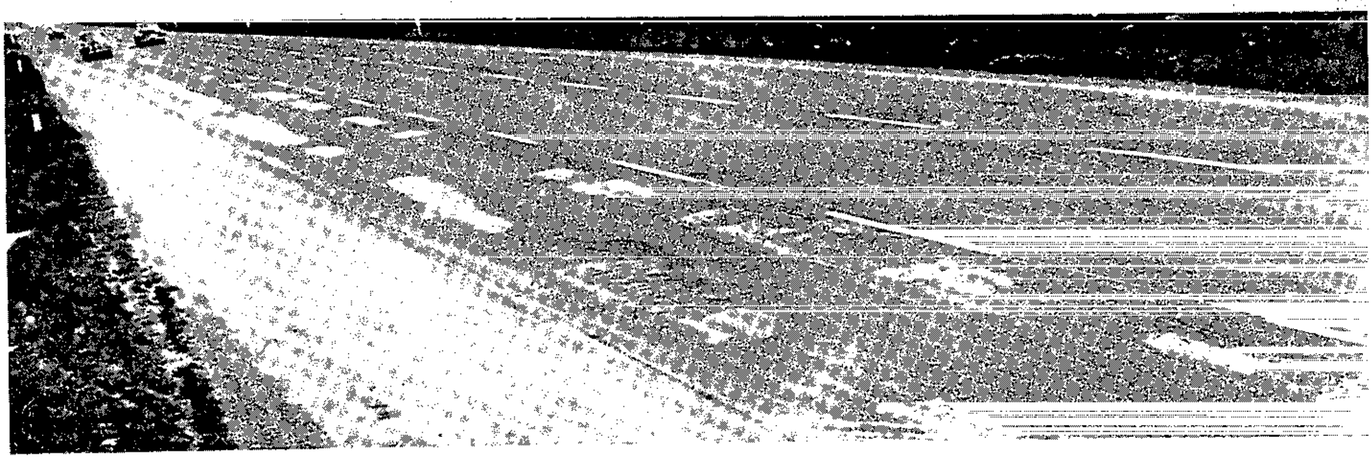
Typical Illustration of
"Chicken Wire" or "Alligator" Cracking

Figure 14 shows numerous areas of distress in the form of cracking and breakup of the pavement in the outer wheel lanes which carry the bulk of the heavy trucks. Deflection measurements showed that the entire foundation was affected by each heavy axle load to a depth of at least 20 feet and that deflections were much higher in the areas showing distress than in limited portions where the pavement was in good condition.

(See Reference #14 for a more detailed discussion.)



Sta. 395± General Outside Lane Failure



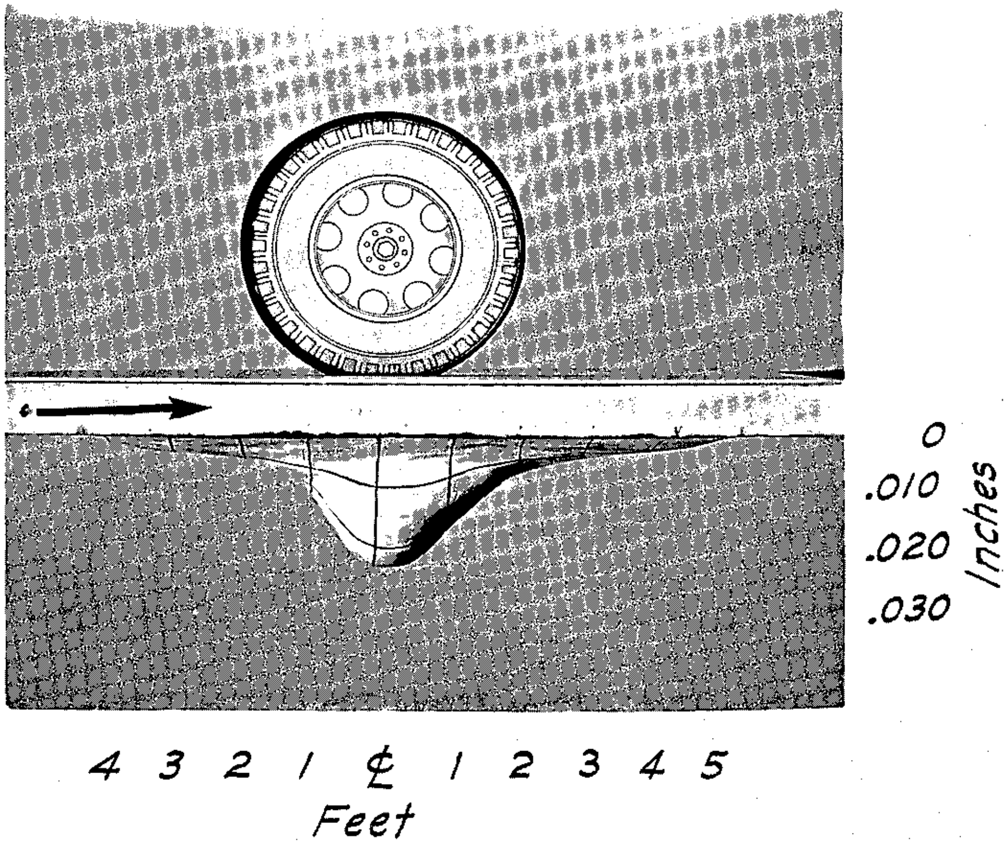
Sta. 498± General Outside Lane Failure

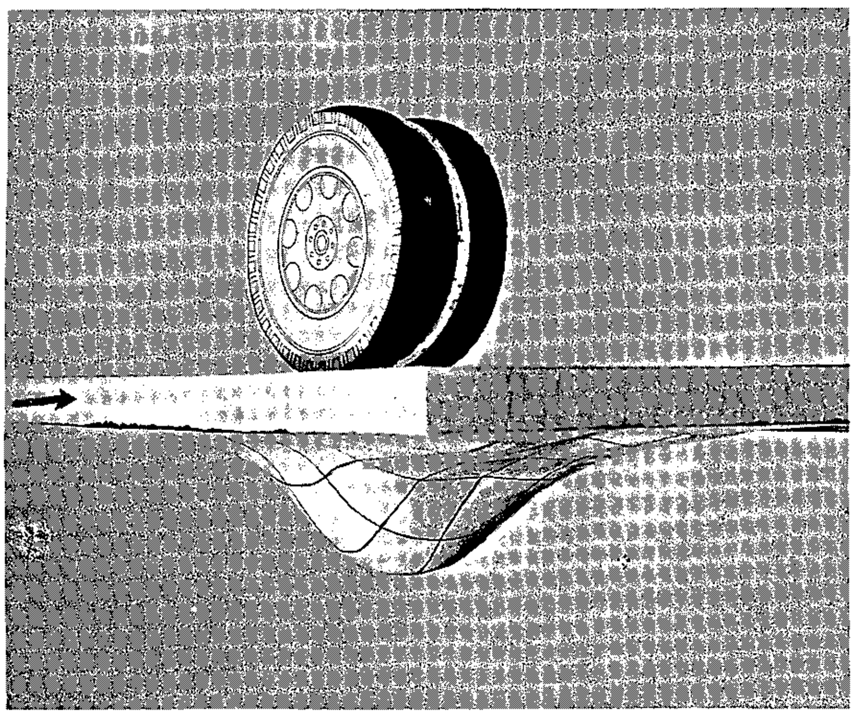
Typical Failed Areas
San Francisco Bayshore Freeway

Figure 15. This is a model carefully constructed from deflection measurements to illustrate the pattern of downward bending or bulging of the underside of an asphalt pavement subjected to a 15,000-lb. dual wheel load. The deflections shown in the illustration appear to be about the limit that a 3" asphalt pavement will withstand for a more or less indefinite period. Any greater deflection than this will apparently produce cracks or "fatigue failures" sooner or later in the average Western asphalt pavement.

(See Reference #14 for a more detailed discussion.)

FIGURE 15





Model Showing Deflection
Pattern Under a Dual Wheel Load

Figure 16. A series of liquid asphalts of SC-4 grade were received from Joseph Zapata, formerly Materials Engineer in the Wisconsin Highway Department. According to reports, samples numbered on the chart as 1, 2, 3, 4 and 5 gave good performance showing no evidence of hardening or deterioration. Sample shown on the chart as Wisconsin No. 7 was reported to have dried up until the road surface virtually disintegrated. The diagram illustrates the results of testing specimens prepared with 2% of the asphalt mixed with Ottawa sand, subjected to an artificial weathering machine equipped with infrared lamps. The horizontal scale represents the number of cycles in the weathering machine. The vertical scale represents the loss in the Asphalt-Ottawa sand test specimen subjected to the abrasive action of a falling stream consisting of 1,000 grams of steel shot.

(See Reference #15 for a more detailed discussion.)

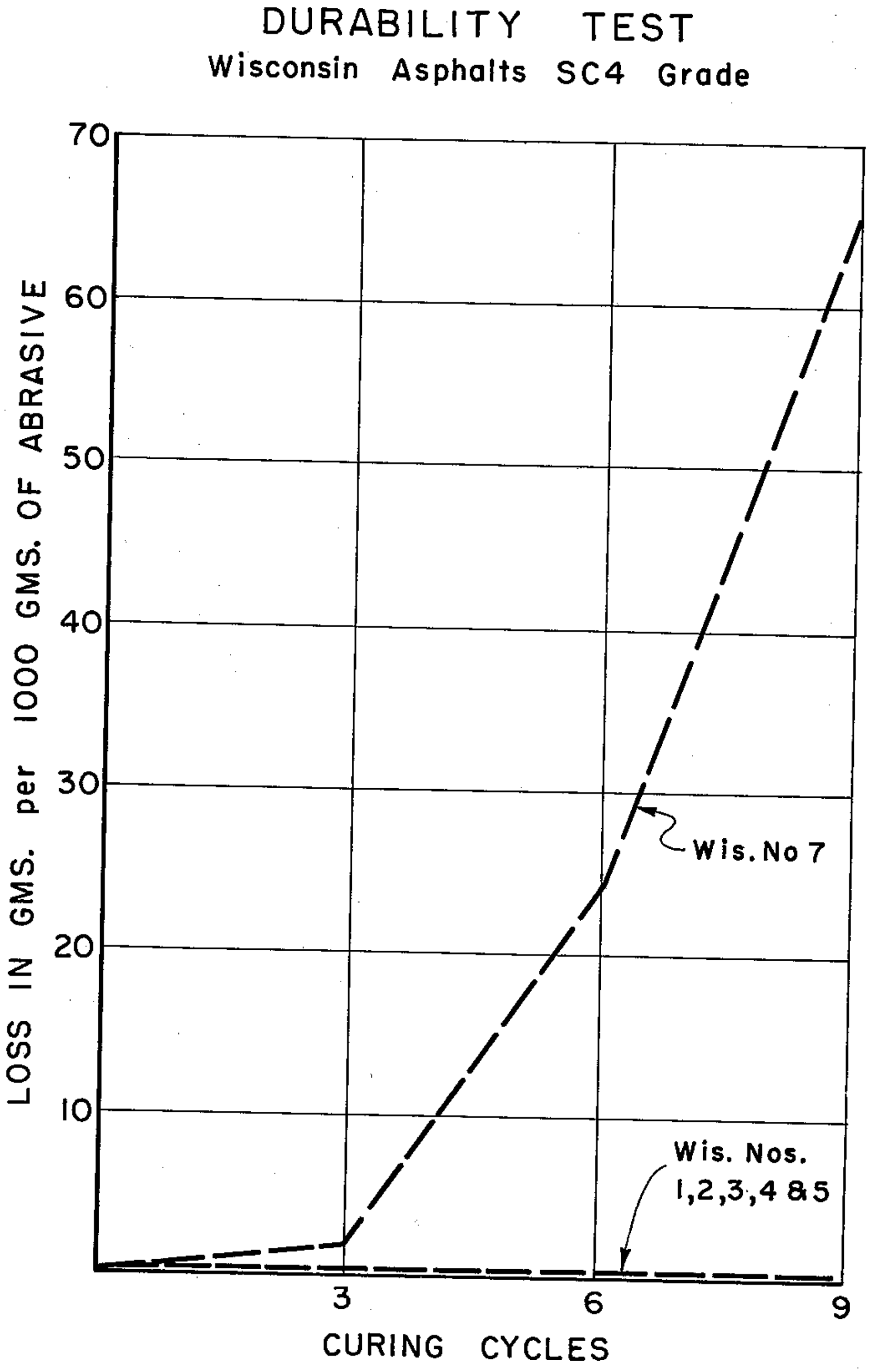
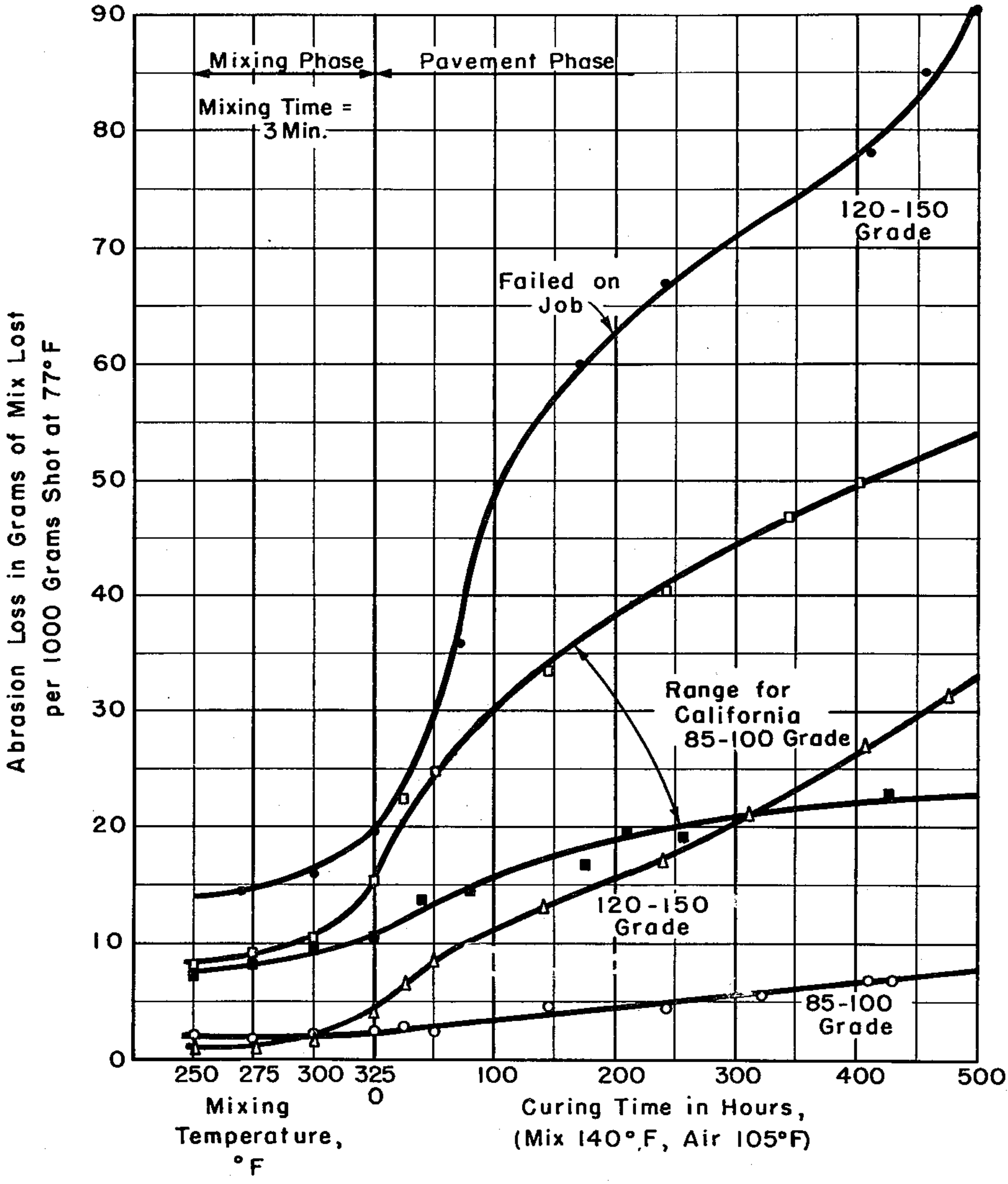

DURABILITY TEST
Wisconsin Asphalts SC4 Grade
70
60
50
40
30
20
10
LOSS IN GMS. per 1000 GMS. OF ABRASIVE
3
6
9
CURING CYCLES
Wis. No 7
Wis. Nos.
1,2,3,4 & 5

Figure 17. This is a chart showing relative durability of paving grade asphalts as developed under conditions simulating normal pug mill or hot plant mixing temperatures and the deterioration in a special weathering machine using infrared lamps to develop temperatures of 140°F. This test is based on the concept that asphalts showing undesirable aging characteristics will develop hardness or brittleness and that compacted samples of Ottawa sand coated with the asphalt in question will show increasing loss in abrasion when subjected to a falling stream of steel shot. Changes indicated by the first four points on the left of the chart are those arising from increasing the temperature at the time of mixing as shown on the abscissa scale. Curves on the right of this line reflect the changes that occur when subjected only to normal atmospheric conditions such as exist in the average pavement during summer months. It is clear that there is a wide variation in the important properties of asphalts meeting current specifications.

(See Reference #17 for a more detailed discussion.)

ABRASION TEST CURVES FOR PAVING GRADE ASPHALTS



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